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TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. LVII.

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AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 1026.

THE CONTROL OF HYDRAULIC MINING IN CALIFORNIA BY THE FEDERAL GOVERNMENT.*

By William W. Harts, M. Am. Soc. C. E.†

With Discussion by Messrs. H. H. Wadsworth, Franklin Riffle, J. D. Galloway, H. DeC. Richards, Stephen E. Kieffer and William W. Harts.

As far back in the past as history takes us, the search for gold has had a passionate interest for mankind. The same zeal that animated the Argonauts who accompanied Jason in his quest of the Golden Fleece, the same patience and perseverance that urged on the searchers for the philosopher's stone, the insatiable greed for quickly acquired wealth that sent the Spanish conquerors to the New World, are even in this day driving men to deeds of daring and adventure at the farthest ends of the earth.

The richness of the treasure discovered in California a generation or more ago seems almost fabulous. Gold fields that yielded in a single year more than $81 000 000, and since the first discoveries have produced up to date more than $1 414 000 000, have filled the minds of an army of miners, from that day to this, with that fever for gold which, once acquired, seems never to relax its hold on its victim.

* Presented at the meeting of May 2d, 1906.
† Captain, Corps of Engineers, U. S. Army.

Scarcely a stream of the western slope of the Sierras but held in its gravel bed quantities of this precious metal, and when all these had been exhausted it was found that enormous wealth could be extracted from the old gravel beds of the prehistoric rivers, those of the tertiary age. To wash through sluice boxes in the quickest time possible these vast quantities of gravel, in order to secure the gold contained, was the eager effort of thousands of men. Millions of cubic yards of these old gravel deposits were thus displaced every year, and washed into the streams and watercourses.

The State of California, not realizing or not caring that some day this might lead to much trouble, assisted the miners in every conceivable way. The right of condemnation of private property for rights of way for mining ditches was granted to the miners, the use of the streams to carry away the tailings of the mines was at one time authorized by legislation, and even the Federal Government, on whose public lands these mines were mainly located, not only made no charge for the privilege of extracting the gold, but even authorized the miners to take up claims without charge, in which they could have a temporary possessory right, protected by law as long as they worked their ground.

It is no small wonder that little thought was given to injuries which might be caused by these operations.

The topography of the Sacramento and San Joaquin Valleys is peculiar, and any interference with the outflow of the floods of these rivers is certain to aggravate a condition already a very troublesome one by nature. Along the flanks of these rivers for miles are rich areas, sometimes several miles wide, which are overflowed each year because of the inability of the rivers to carry off the floods caused by the spring rains and melting snows. For months these rich lands are under water. As the population of the valleys along the rivers increased, a growing hostility arose toward the miners who, it was believed, were mainly responsible for these conditions. Thus arose the historic contention between the miners on the one hand and the valley people on the other, which led to much bitterness of feeling in later years. Its settlement has occupied the minds of many engineers and legislators for years, as well as the time of many lawyers and judges. The end is only now barely in sight.

The Debris Problem.—The problem of what to do with the debris

resulting from years of unrestricted hydraulic mining has thus vexed the people of California for upwards of a generation. In the fierce and bitter conflict between the mining interests of the mountains on the one hand, and the interests of the agricultural regions on the other, which has been carried on for more than thirty years, the industry of hydraulic mining was finally completely vanquished. The decisions of the courts, and their injunctions, had put an end by about 1880 to a form of mining activity in which more than $100 000 000 were said to have been invested at that time, and from which the State of California had originally derived most of its claim to the attention of the world. These decisions and injunctions against hydraulic mining were based on the injuries caused by the tremendous inundation of mining detritus over the farming lands and river beds of the valleys of the central part of the State, and on the constitutional right of the owners of this property to be protected from such injuries.

It cannot be denied, however, that the discovery of gold in California in 1848 gave a stronger impetus to the growth of the State than any other single occurrence before or since. By 1849 there were 100 000 miners engaged in washing the gravel of the river beds of the Sierras. Where a sparse and scattered population had worked out a more or less precarious existence, the discovery of gold caused mining camps first and then flourishing towns to spring up as if by magic. As these grew, the mining industry was followed by the farmers, orchardists and ranchers who settled later in the rich valleys of the Sacramento and San Joaquin Rivers. To save their farms from inundation caused by raising the flood plane of the rivers, and to prevent the bottom lands from being ruined by the accretion of gravel and debris from the mines in the mountains and foothills, legal remedies were resorted to by the valley people, and by about 1880 the once enormous industry of hydraulic mining had been successfully throttled by the injunctions of the Local and Federal Courts.

The feeling between the miners and the valley people became acute, and much bitterness resulted. The State of California had extensive investigations made, but found it could not cope with the difficulty. The Federal Government was then appealed to, and examinations were ordered by Congress in 1880 and again in 1888.

The former was made and a report submitted by Colonel G. H. Mendell,* Corps of Engineers, U. S. Army. The latter was made by a board of three officers consisting of Lieutenant-Colonel Benyaurd, Major Handbury and Major Heuer, all of the Corps of Engineers.† On the report of this board, Congress passed, in March, 1893, a mining law which has been since known as the Caminetti Act, from the name of its proposer in Congress. This Act was ostensibly for the protection of the navigability of the Sacramento and San Joaquin Rivers, by preventing them from being filled with mining debris and their depths thus reduced to a point where free navigation would be impossible. It was fervently hoped by those interested in mining in California that the application of the law would be far wider in its benefits and give them the long-sought privilege of operating their mines without hindrance. This law created a Federal commission to regulate hydraulic mining in such a way as to prevent any injury to navigable waters, with ample power to stop all such mining until satisfactory impounding facilities for its detritus were provided by each mine.

The constitutionality of such an enormous interference with the interior affairs of one of the States of the Union as was embraced in this law was based on the provision of the Constitution of the United States assigning to the Federal Government the duty of regulating the commerce between States, and incidentally of protecting the navigable highways.

It was presumably understood in Congress that the steps necessary to protect the interests of navigation from the mining debris evil would necessarily protect all the agricultural interests, as the greater includes the less, so that if the engineers in charge could devise a method which would protect satisfactorily the navigability of the rivers affected, the other questions involving private rights would solve themselves. The depths of the rivers being affected so quickly, there was thus afforded a delicate test of injury. Any mining that would cause no appreciable shoaling in the navigable rivers could scarcely be of any injury to any other interests. If the navigability of the rivers was protected, therefore, the problem that had been of such long standing could be soon settled. This law, it was

* Ex. Doc. 76, House, 46th Cong., 3d Session.
† Ex. Doc. 267, House, 51st Cong., 2d Session.

thought, would thus permit the resumption of hydraulic mining under such restrictions as might be necessary to protect the rivers.

The Act provides for a Federal board of three engineer officers of the army, appointed by the President and confirmed by the Senate, to be called the California Debris Commission. It became their duty under the law to devise plans that would permit hydraulic mining to be resumed, under sufficient restrictions to prevent injury to the lower rivers and the land lying adjacent thereto. The Commission's jurisdiction was limited to the water-shed of the Sacramento and San Joaquin Rivers, but within these limits all hydraulic mining was prohibited except with the license or permit of this Commission. A maximum penalty of $5 000 fine and one year's imprisonment was imposed for violation of the Act.

The Commission was directed also to devise projects that would improve the navigability of the rivers above named and their tributaries, protect their banks against encroachment and damage from mining debris, and restore, as far as required by the needs of navigation, the navigability of the rivers as it existed in 1860.

What has been accomplished by this Commission since its organization is the subject of this paper.

Description of the Mining Region.—To understand fully what was assigned to the California Debris Commission and the wide scope and difficulty of its work, the geography and topography of the State of California must be considered briefly in connection with the flow of mining debris.

The entire central portion of the State is inclosed by mountains, and consists of an elliptically shaped valley, about 450 miles long and 40 miles wide, embracing about 18 000 sq. miles. It is hemmed in by the Coast Range on the west and the Sierras on the east, and is drained by the Sacramento and San Joaquin Rivers, which empty into the Pacific Ocean through San Francisco Bay. All the water falling on the areas of the water-shed of these rivers must pass into the Pacific Ocean through one mouth, about a mile wide at its narrowest point, called the Golden Gate.

The flat portion of the valley embraces about 4 769 sq. miles, where the annual rainfall is about 18 to 20 in.; the Sierra slope, with elevations as high as 11 000 ft., has an area of about 8 843 sq. miles, where the rainfall varies from 24 to 102 in.; the slope of the Coast

Range has an area of 3 075 sq. miles, with a rainfall of something less than the Sierra slope, whereas the Shasta region on the north, having elevations of more than 14 000 ft. and an area of 5 616 sq. miles, has a rainfall ranging from 30 to 110 in. The rainfall over the entire water-shed of the Sacramento Basin, of about 26 187 sq. miles, may be averaged at about 30 in.*

Practically all this rain falls during the winter months and in the early spring, so that the warm rains and melting snows together frequently cause excessive floods. The watercourses in their upper portions are all mountain torrents. It thus happens that every spring the Sacramento River is required to carry off floods far beyond the capacity of its bed.

The superficial area of the channel and slough surface is only 38 sq. miles, out of a total of 4 769 sq. miles of the level part of the valley, and the fall in surface slope is very limited in the lower portions of the river. It has been estimated that, with an annual precipitation of from 30 to 40 in., there will be an area of 1 700 sq. miles, more or less, flooded every spring when a warm rain sets in after heavy snows.†

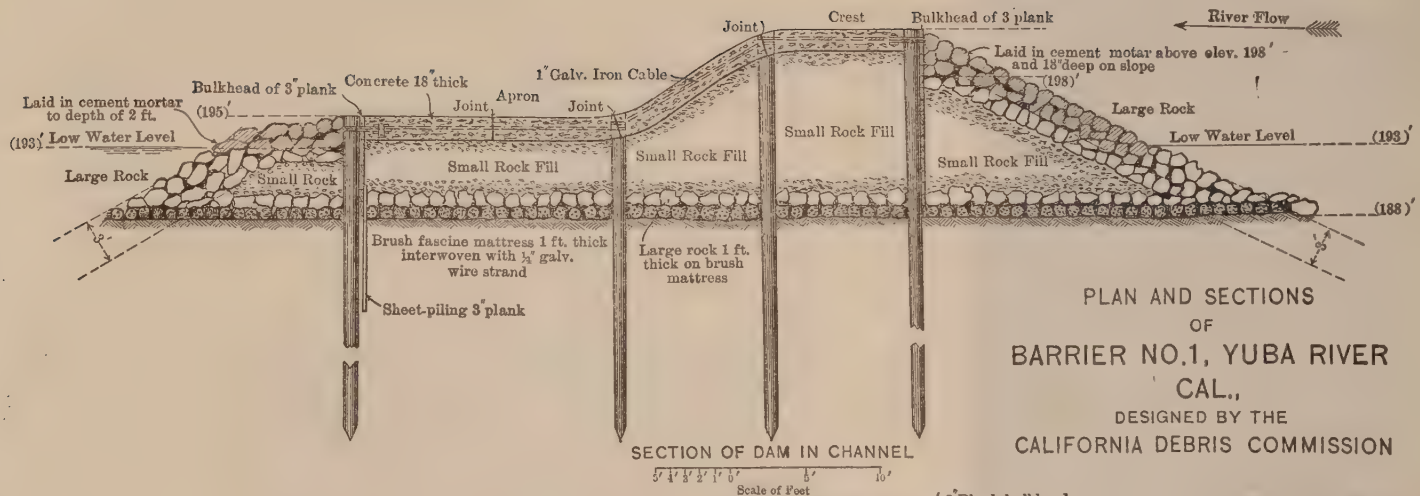
It is very evident that the local conditions have for ages, if not always, prevented the Sacramento River from discharging its flood waters promptly. Furthermore, for ages the well-known erosive action and leveling tendencies of all rivers have been at work cutting down the mountains and hillsides and filling up the lower river reaches and the bays. Much of the Sacramento Valley has unquestionably been formed in this way, and this action is doubtless still in progress. It seems more than likely that Suisun Bay is all that is left of a large inland sea, the waters of which at one time probably washed the lower slopes of the Sierras.

The denudation of various valleys by natural causes in different parts of the world progresses at different rates, and it is unfortunate that no definite records are available for the Sacramento water-shed.

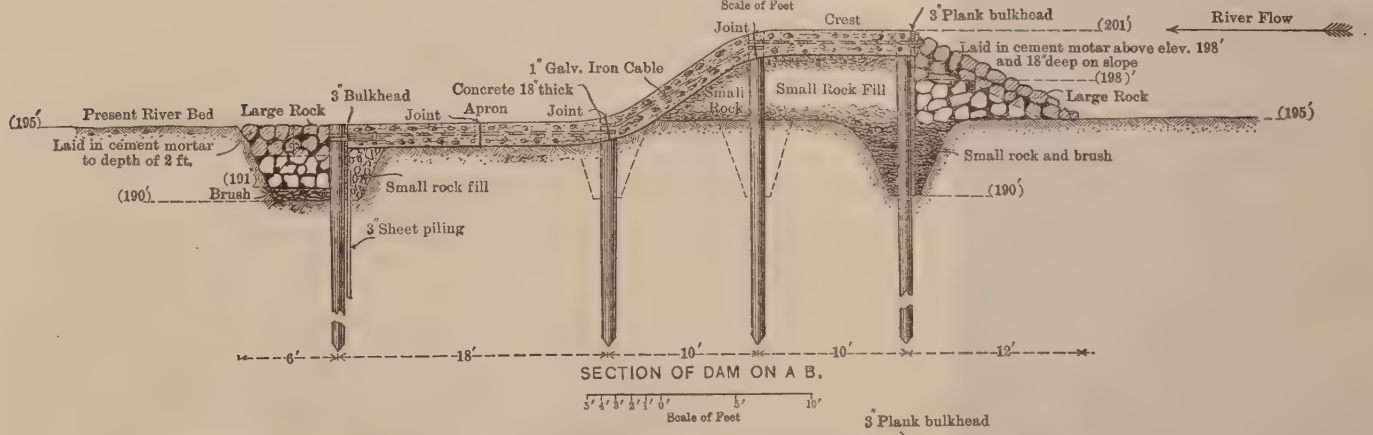
Observations on the quantity of sediment carried by various rivers in various lands indicate that the time required by natural agencies to remove an average thickness of 1 ft. of rock in each of the several drainage basins is as follows:

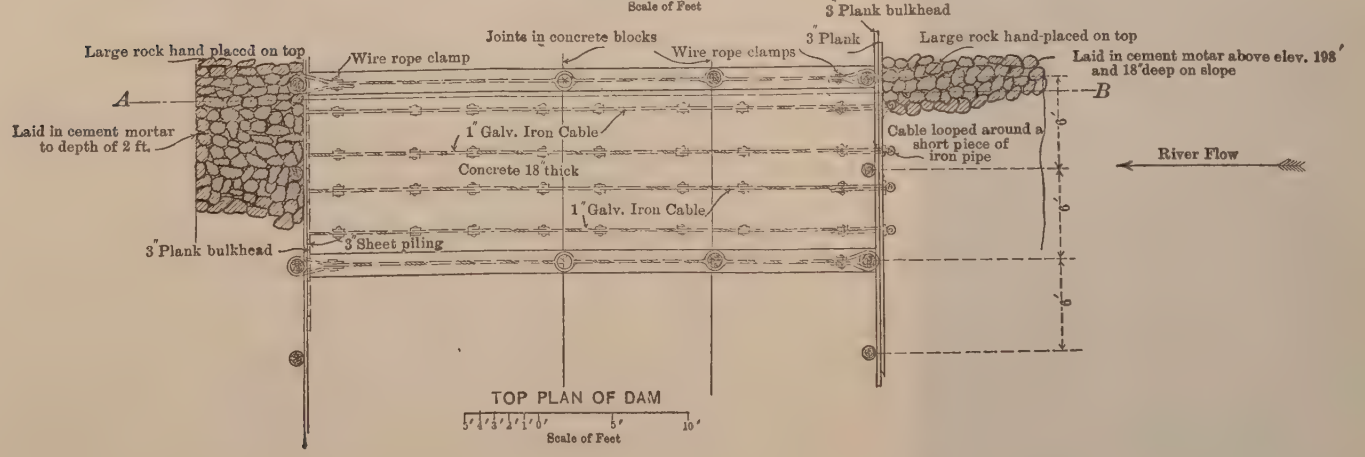
* "Physical Data and Statistics of California," by William Ham. Hall, M. Am. Soc. C. E.

† *Transactions*, Technical Society of the Pacific Coast, February and March, 1887.



PLAN AND SECTIONS
OF
BARRIER NO. 1, YUBA RIVER
CAL.,
DESIGNED BY THE
CALIFORNIA DEBRIS COMMISSION




3" Plank bulkhead
Joints in concrete blocks
3" Plank
Large rock hand placed on top
Wire rope clamp
Wire rope clamps
Large rock hand-placed on top
Laid in cement motar above elev. 198' and 18" deep on slope
A
B
Laid in cement mortar to depth of 2 ft.
1" Galv. Iron Cable
Cable looped around a short piece of iron pipe
River Flow
Concrete 18" thick
1" Galv. Iron Cable
3" Plank bulkhead
3" Sheet piling
TOP PLAN OF DAM
Scale of Feet

Danube	Basin	6 846 years.
Mississippi	"	6 000 "
Nile	"	4 723 "
Ganges	"	2 358 "
Rhone	"	1 528 "
Hoang Ho	"	1 464 "
Po	"	729 "

Applying the Mississippi rate to the Sacramento River, it would indicate a removal of about 5 600 000 cu. yd. of sediment per annum; at the Po rate it would indicate 43 000 000 cu. yd. The natural conditions of the Sacramento Basin resemble those of the Po more than those of the Mississippi. It is stated as probable that the annual sediment moved by natural causes is somewhere between the rates of these two rivers, that is, between 5 600 000 and 43 000 000 cu. yd. per annum for the Sacramento Basin.*

Nearly all this natural wash is carried into the lower sections of the tributary rivers, some to be transported into the Pacific Ocean with the tidal currents, some to settle on the shoals and tide flats around San Francisco Bay, and some to add to the already gorged condition of the river channels.

We thus see that for probably thousands of years natural agencies have been at work filling up the navigable rivers and encroaching on the area of San Francisco Bay. These natural agencies, already promising certain disaster to the navigable capacity of San Francisco Harbor at some far-distant day, were assisted at an enormous rate by the work of Man, for there were recently added human agencies that rendered more complicated an already too complex situation. This was the removal by hydraulic mining of millions of cubic yards of gravel and sand from their location on the western slopes of the Sierras and the washing of this material into the cañons and tributaries of the Sacramento, Feather, American and San Joaquin Rivers.

Hydraulic Mining Methods.—Gold was discovered in California in 1848, during the construction of a mill race at Colona, near Georgetown, purely by accident. The news spread rapidly, and in 1849 there followed an army of 100 000 miners, who soon washed out

* Annual Report, Chief of Engineers, U. S. Army, 1882.

all the accessible gravel lying in the beds of the small creeks and rivers.

The hand-pan, in which only a small quantity of gravel at a time could be washed, gave way early to rockers of larger capacity, and these to sluices where riffles caught the gold, after being washed and separated by the action of flowing water, brought in ditches, sometimes for great distances. It was only a step further to the use of hose and nozzles, and then to steel pipes and monitors for directing powerful streams against the tertiary gravel banks of higher elevations. The streams are used under pressure to break down the banks, and the water carries off the gravel through the sluice boxes, where the gold, by virtue of its high specific gravity, is caught in riffles of various kinds.

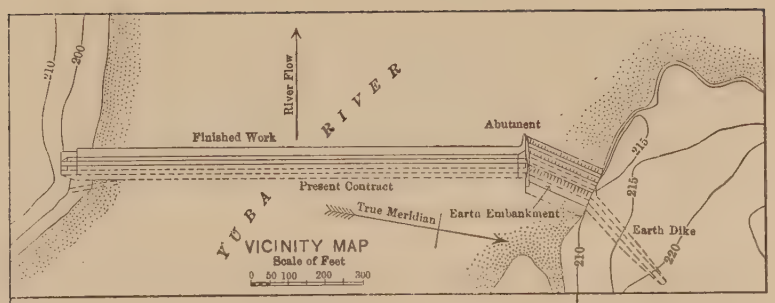
This method was invented by a native of Connecticut, and dates from 1852. It dispenses with many men, and is a cheap and quick method of excavating gravel banks. The cost, in favorable situations, has been stated to have been as low as 3 to 6 cents per cu. yd. Nozzles as large as 9 in. in diameter have been used, pressures as high as that due to 400 ft. head, and quantities as great as 3 500 miners' in. (about 87 cu. ft. per sec.) in a single stream. The excavating power of these large streams is enormous.

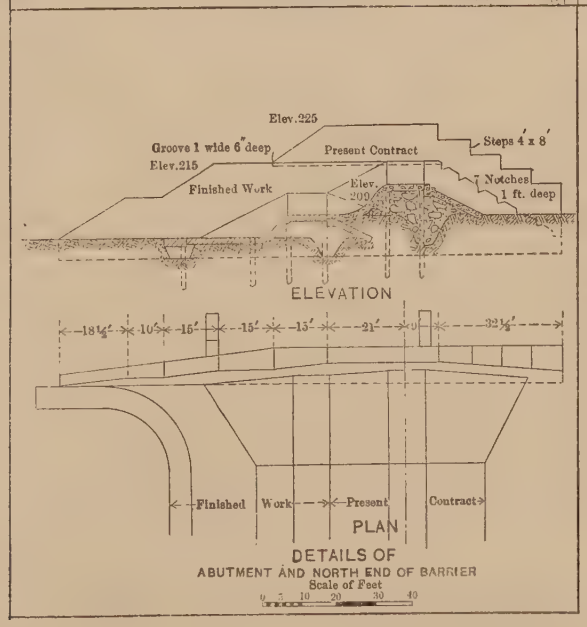
The engineering features connected with these early mining operations are full of interest even now. The difficulties were tremendous, in building mining ditches carrying several thousand inches of water for sometimes 40 and 50 miles along the crests of ridges and over deep gulches in wooden flumes, all to bring the quantity of water necessary to mine the gold, at an elevation sufficient to give the pressures that were indispensable. In securing outfall at the mines sufficient to remove the debris while mining, tunnels of great length, in rock, were also often necessary. These operations were all on a scale but little realized by those who have not visited the mining regions.

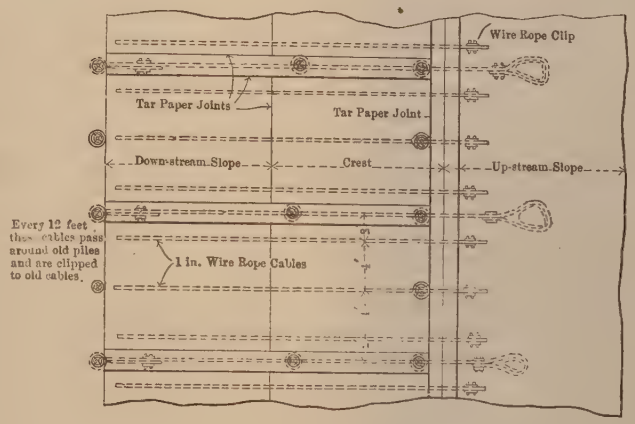
Extent of the Damage.—The quantity of gravel which can be moved in 24 hours by each miners' inch of a stream is called its "duty." This depends on the character of the bank, and varies from ½ cu. yd. to 6 cu. yd., with an average of about 3 cu. yd. This duty, with an approximate knowledge of the total quantity of water used in a season throughout the entire mining field, gives some

PLATE II.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVII, No. 1026.
HARTS ON
CONTROL OF HYDRAULIC MINING.

PLAN AND SECTIONS
OF
SECOND STEP
BARRIER NO.1 YUBA RIVER CAL.
DESIGNED UNDER THE DIRECTION OF THE
CALIFORNIA DEBRIS COMMISSION



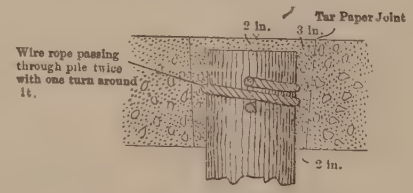



Wire Rope Clip
River Flow
Tar Paper Joints
Tar Paper Joint
Down-stream Slope
Crest
Up-stream Slope
Every 12 feet these cables pass around old piles and are clipped to old cables.
1 in. Wire Rope Cables

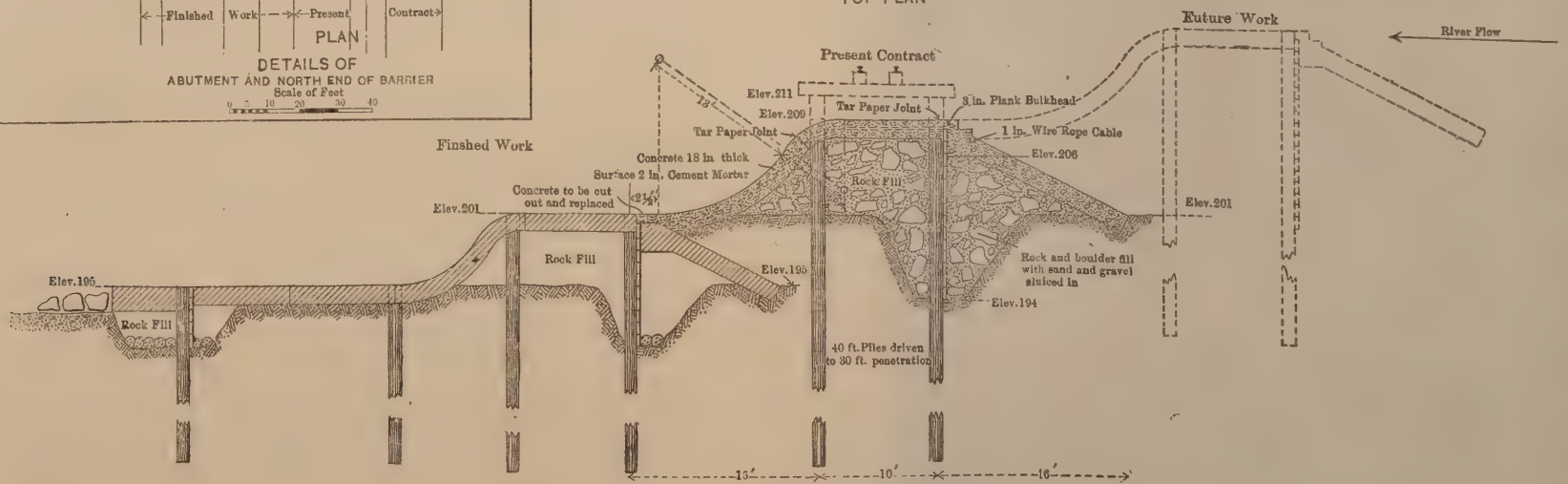
TOP PLAN

Intermediate cables pass 5 feet into up-stream concrete with clip on end.

Up stream end of cables every 12 feet passed around large rock and clamped.



DETAIL OF PILE HEAD
SHOWING CABLE FASTENING END JOINTS
BETWEEN CONCRETE BLOCKS



Finished Work

SECTION OF NEW YORK
SHOWING CONNECTIONS WITH EXISTING WORK

idea of the quantity of material dislodged annually. In this way it was estimated in 1880 by the State engineers that 53 404 000 cu. yd. of gravel were washed from their banks into the cañons and rivers of the Sacramento Basin. Much of this material was coarse, and was not carried far, but most of the fine material was unquestionably taken down by the first freshet, to be added to, year after year, and carried farther at each high water.

Sediment observations of the water of the Sacramento River were made in 1880, and deductions from them indicated that 18 100 000 cu. yd. were brought down that river in suspension that year, of which amount 4 900 000 cu. yd. were assumed to be the proportion due to natural wash and 13 200 000 cu. yd. that due to hydraulic mining. Of the total, 2 100 000 cu. yd. were estimated to have settled in the overflowed basins along the river, leaving 16 000 000 cu. yd. that were carried in suspension past Sacramento and down into Suisun and San Francisco Bays. Of this amount (16 000 000 cu. yd.), it is assumed that 11 100 000 cu. yd. is the proportion due to hydraulic mining. Deducting this 11 100 000 cu. yd. from the total amount excavated by the hydraulic process (53 404 000), as ascertained by the "duty" of hydraulic streams, and referred to above, we find that the remainder, 42 304 000 cu. yd., must have been left behind in the numerous cañons and tributaries of the Sacramento and San Joaquin Rivers as a result of the mining operations of a single year.*

These conclusions are based on what now appears to be insufficient observation, and the reference to them here is mainly intended to give an indication as to the enormous extent of the debris problem. A visit to the mining regions, for example, from Nevada City to Dutch Flat, would dispel any doubts in anyone's mind that the denudation by this means was enormous.

The damage done to the streams and lower country is as palpable and manifest to anyone who has visited the Yuba, the Bear and American Rivers as is the extent of denudation.

The vast quantity of debris now lying in the river-beds has been distributed along the watercourses in accordance with the well-known laws of water transportation. Much of it was undoubtedly left where new conditions of flood would move it again to a new

* Report, State Engineer, 1880.

location lower down. It thus seems to be plain that the enormous quantities of tailings in the beds of the cañons and streams are not permanently located, but are only awaiting an opportunity to move to a new location whenever the velocities of the stream are favorable. The injury once done still exists as a menace for the future, and will continue to exist until conditions of more perfect equilibrium are reached. Surveys recently made indicate that between 1899 and 1904 an addition of more than 15 000 000 cu. yd. was lodged in the bed of the Yuba River from these old deposits.

The low-water plane of the Yuba River at Marysville was raised 15 ft. between the years 1849 and 1881.* The citizens of Marysville, in order to protect their town from the damage due to floods occasioned thereby, built levees around the city, which were added to from time to time as required. These levees were continued up to Daguerre Point, about 11 miles, on the north bank. This necessitated levees on the south side of the river, to protect the adjoining agricultural lands, and thus grew up an extensive levee system which is now maintained at great expense. The lower river, from a narrow mountain stream in 1850, is now filled with sand and gravel so that its bed is nearly 3 miles wide in places, with an average width of about 2 miles. This abnormal width, lying between the lines of levees, extends up the river from Marysville to Daguerre Point, in all about 9 miles. About 25 sq. miles are covered with mining debris. The depth of fill varies from about 7½ ft. at Marysville to 26 ft. at Daguerre Point and 84 ft. at Smartsville. A short distance east from Marysville the bed of the river is now 13 ft. above the level of the surrounding farms. It is plain that any accident to the levees near Marysville would mean a disaster to the town.

The quantity of material lodged in the river due to mining has been variously estimated at from 71 000 000 to 700 000 000 cu. yd., but it seems safe to say that there are now upwards of 333 000 000 cu. yd. in the bed of the lower Yuba. This material varies in character from cobbles, near Smartsville, to an impalpable powder near Marysville, being distributed along the river-bed from coarse to fine as the slopes diminish. These slopes decrease from about 15 ft. per mile near Smartsville to about 5 ft. per mile at Marysville.

* Appendix MM, Annual Report, Chief of Engineers, U. S. Army, 1882.

Not even a faint conception can be had of the vast body of mining material in this river, without seeing the river-bed at low water.

The Bear River shows greater depths of mining detritus than the Yuba, but the quantities are not as large, the cañon of the Bear being steeper and narrower. The best information available leads to the conclusion that the Bear River is filled 150 ft. deep at the crossing between Little York and You Bet. The quantities lodged in the Bear River were roughly estimated at 66 000 000 cu. yd. in 1891.*

The Feather, the American, Consumnes, Calaveras, Mokelumne and other tributaries of the Sacramento are all, in greater or less degree, affected in the same way.

The Sacramento River itself shows unmistakable signs of considerable fill, largely due to mining detritus. Since 1849 the low-water plane at Sacramento has been raised about 7½ ft., causing a reduced carrying capacity for its flood waters, and requiring property owners to build levees to protect themselves from floods. The available depth for navigation, however, has remained about the same. The quantity of debris in the river is estimated at 108 000 000 cu. yd.†

A comparison of charts shows that the bar across the entrance to San Francisco Bay has not deteriorated noticeably, nor the channels within the bay diminished in depth, but shoaling on the areas along the flanks of the channels is perceptible, and the channels are slowly narrowing. In Suisun Bay the area of shoals is increasing, and at Carquinez Straits there are indications of the formation of large deposits.‡

Duties of the Commission.—Such, then, was the condition that confronted the first Commission in 1893. To check the injury to navigable streams at once, to prepare plans for the improvement of their navigability by restraining works, and to rehabilitate hydraulic mining as far as practicable, were the duties imposed upon this Commission by the Act. The supply of new detritus from hydraulic mines had by that time been mainly, if not entirely, stopped by injunctions of the Federal and State Courts, prohibiting all such mining, but the flow of debris was still continuing, due to the washing

* Appendix MM, Annual Report, Chief of Engineers, U. S. Army, 1882.
† Ex. Doc. No. 267, House, 51st Cong., 2d Session.
‡ Appendix MM, Annual Report, Chief of Engineers, U. S. Army, 1882.

down at every flood of a part of the enormous quantities already stored in the upper river reaches and the tributary cañons. The damage once started could not be wholly overcome by injunctions.

In designing restraining works there was no precedent nor previous experience from which valuable lessons might be learned by the new Commission, but everything had to be originated *de novo,* as no such condition probably exists elsewhere in the world.

In addition to all this, the miners were clamoring for permission to resume mining under the new law, insisting that they be told of the restrictions so that they might comply with the law, if found practicable, and be permitted to resume their business legally.

The first efforts of the Commission were thus directed to making examinations of the mining regions, in order to see what relief could be given the miners.

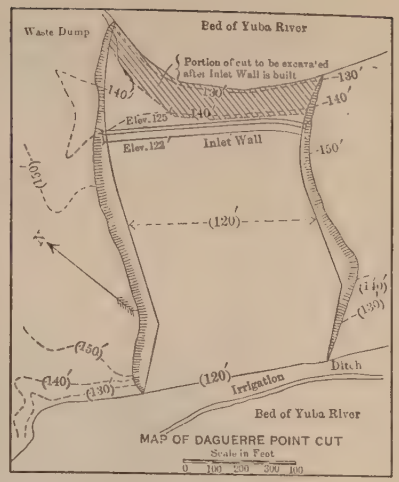
It was found in many cases that the construction of dams in the cañons below the mines would store all the material that would be moved, and in some cases there were old hydraulic mine pits that could be filled with detritus without damaging the lower rivers. Other expedients were studied, but the method of using dams across the ravines and cañons below the mines was found most satisfactory for the general case.

Impounding Dams.—Various kinds of dams were tried: stone, earth, brush and rock, log-crib filled with rock, and many others. After twelve years of experience it has been found that the usual small mine where impounding dams can be used will need one of two general types—either log-crib dams or brush dams.

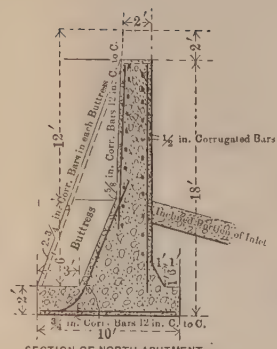
There are special cases, of course, where other kinds of dams are needed, but these two types are most common for the smaller mines, and printed specifications for these dams, with a cut explaining their construction has been prepared by the Commission.

The log-crib is the usual type. It consists of a "cob-house" crib made of large logs which are notched and drift-bolted together. It is filled with quarried rock and chinked against leakage. This type of dam is seldom built more than 40 ft. high, this being the limit of safety placed by the Commission for the usual case. These dams are very satisfactory for their purpose when well made. As long as they are kept wet they are practically permanent, and in those locations where the logs rot, due to being dry part of the time, the rock

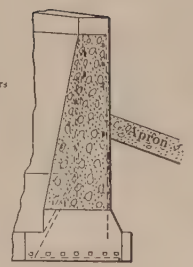
PLANS FOR INLET WALL AT DAGUERRE POINT CUT, YUBA RIVER, CALIFORNIA, FOR CALIFORNIA DEBRIS COMMISSION



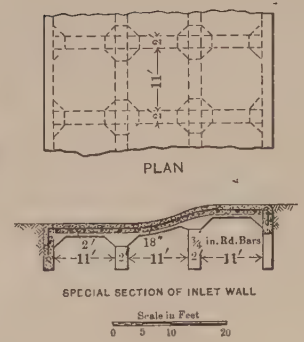
MAP OF DAGUERRE POINT CUT
Scale in Feet



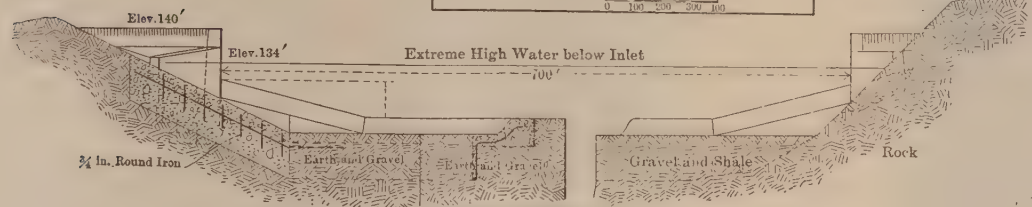
SECTION OF NORTH ABUTMENT
PROBABLE MAXIMUM HEIGHT 18 FT
DESIGNED FOR HEIGHT OF 20 FT.
Scale in Feet



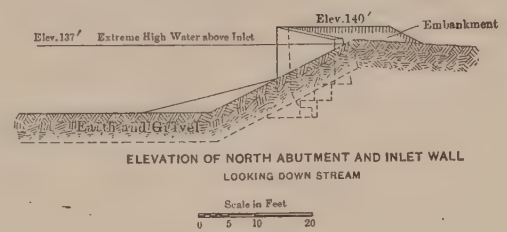
SECTION ON E-F
Scale in Feet



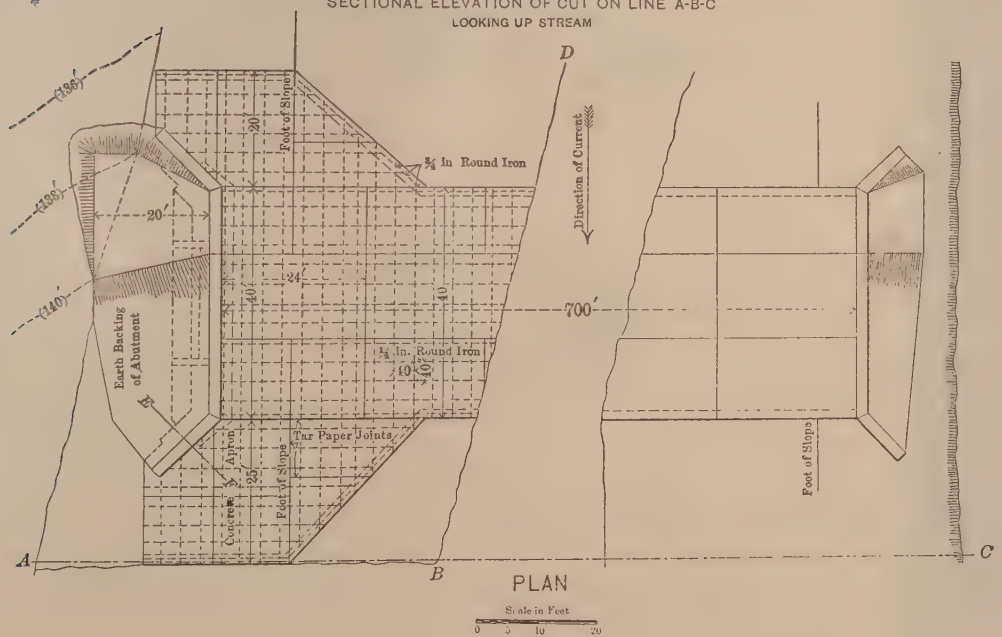
SPECIAL SECTION OF INLET WALL
Scale in Feet



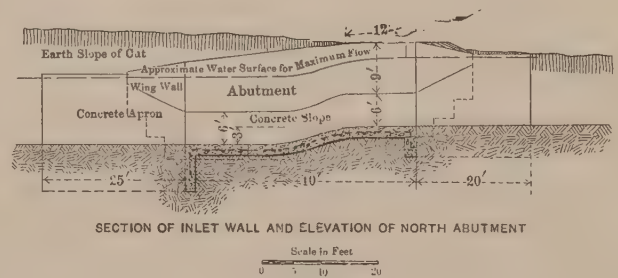
Elevation on Broken Line B-D
SECTIONAL ELEVATION OF CUT ON LINE A-B-C
LOOKING UP STREAM



ELEVATION OF NORTH ABUTMENT AND INLET WALL
LOOKING DOWN STREAM
Scale in Feet



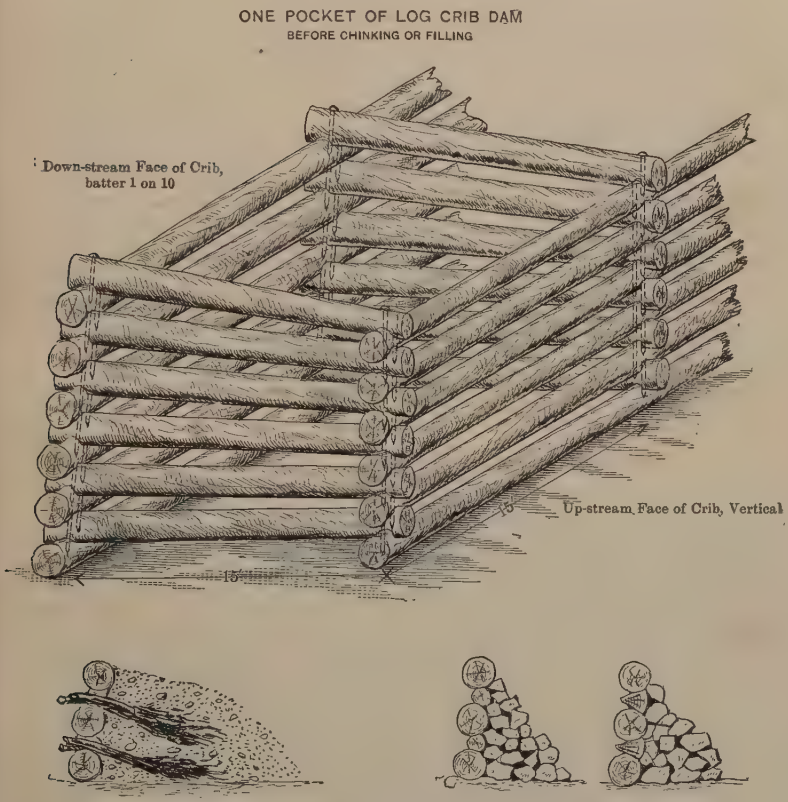
Scale in Feet



SECTION OF INLET WALL AND ELEVATION OF NORTH ABUTMENT
Scale in Feet

being angular and well bedded in gravel will resist erosion long after the logs have failed to bind the dam together.

The brush dam is less used, as it is permitted usually only when the water-flow over the dam is small, or when the river is diverted


ONE POCKET OF LOG CRIB DAM
BEFORE CHINKING OR FILLING
Down-stream Face of Crib, batter 1 on 10
Up-stream Face of Crib, Vertical
15
15

SKETCHES SHOWING METHODS OF CHINKING DOWN-STREAM FACE

FIG. 1.

through a spillway at one end, and only when the slope of the cañon above is slight. These brush dams are not permitted more than 20 ft. in total height.

Figs. 1 and 2, with the specifications and the photographs herewith will show plainly their construction in both cases.

General Instructions for Log-Crib Debris Dams.

Approved by the California Debris Commission, January 18th, 1904.

1.—The bottom and sides of the dam are to be founded on bed-rock, and the ends of the timbers set into bed-rock wherever practicable, so as to provide a shoulder against which the dam may rest to resist the pressure of the debris when impounded.

2.—The dam will consist of a down-stream and an up-stream wall of logs connected by cross-logs running up and down stream, the walls of cross-logs to be not more than 16 ft. apart.

3.—All logs are to be as large as practicable, and to be well notched and drift-bolted together at crossings.

4.—The distance between the up-stream and down-stream walls of logs is to be not less than one-half the proposed finished height of the dam, and in no case less than 15 ft.

5.—The up-stream wall is to be vertical, and the down-stream wall is to have a slight slope up stream of about 1 ft. in every 10 ft. in height.

6.—The spaces between the logs in the down-stream wall are to be closed by small logs laid inside the dam, or by brush, as shown in Fig. 1.

7.—The dam is then to be filled with stone and chinked with fine brush, leaves, etc., so that while mining is in progress, it will maintain a pool of water at least 2 ft. deep.

General Instructions for Brush Debris Dams.

Approved by the California Debris Commission, January 18th, 1904.

1.—Brush dams should be built of live strong brush at least 10 ft. long. All large limbs should be hacked with an axe, but not cut off, and then bent back to lie compactly. Small twigs and leaves should be left on. The poles used should be not less than 4 in. in diameter and not more than 12 in. The poles should be well trimmed, and as long as practicable.

2.—The dam should be built along a straight line, as follows: Level off the foundation. On this lay the brush closely, with the butts in a line and pointed down stream. This should make a thick, compact layer. On top of this layer and at right angles to the brush lay a pole about 2 ft. back from the ends of the butts, which, with other poles like it, should extend entirely across the stream.

3.—A layer of gravel or small stone is then placed on the layer of brush as high as the thickness of the pole. On this layer of

FIG. 1.—BANK OF A HYDRAULIC MINE (NOT IN OPERATION) IN NEVADA COUNTY, SHOWING GREAT DEPTH OF EXCAVATION.

FIG. 2.—DISHONEST SLUICE, WITH CONCEALED TRAP, PARTLY OPEN, TO PERMIT TAILINGS TO PASS INTO A SIDE RAVINE, INSTEAD OF INTO THE AUTHORIZED SETTLING BASIN.

gravel place another heavy, compact layer of brush as before, butts down stream and tips up stream, on which lay another row of poles across the stream. Then place another layer of gravel as before, and so continue until the dam is of the required height. The dam should then consist of alternate thick layers of brush and thin layers of gravel, each two layers of brush separated by a row of poles. See Fig. 2.

4.—The poles should be placed so that each row is somewhat back of the row below, so that the whole down-stream face of the dam when completed will have a slope of about 3 horizontal to 4 vertical, and so that the butts of the brush will be about 2 ft. higher than the tips. Each row of poles should be strongly wired, every 4 ft., to the row of poles below.

5.—The dam must be tightened against leakage, with gravel and fine brush thrown on the tips of the brush, so that, when the

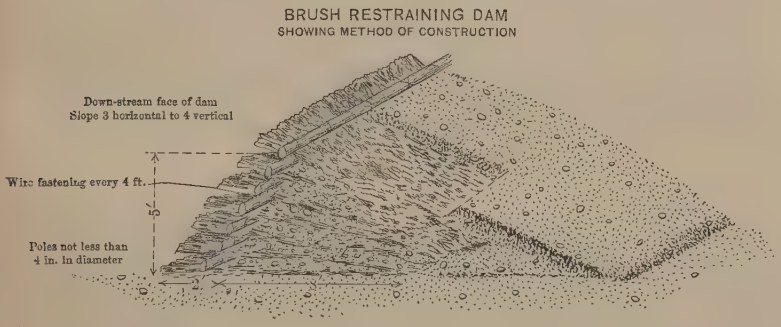


FIG. 2.

mine is being worked, a pool of water at least 2 ft. deep will be always maintained.

Requirements for a License.—Before being permitted to mine, under the United States laws, the hydraulic miner must have a license, or permit, from the California Debris Commission. In order to obtain this license, he must submit an application, or petition, on the form supplied by the Commission, in which he states under oath the location and description of his mine, its extent, the source and quantity of his water supply, the dimensions and grade of his sluice boxes, what restraining works he proposes, the precipitation of rain and snow, the drainage area above his mine, and several other items of information which affect the flow of detritus. This application is advertised in newspapers for three weeks, to permit any protests

to be filed with the Commission. If no protests are received, an inspection of the mine and its impounding facilities is made, and the location and kind of restraining dam decided upon in consultation with the owners of the mine. An order is sent by the Commission to construct the restraining works decided upon, and later another inspection is made when the works are completed. If these works are found satisfactory on this second inspection, a revocable license to mine is issued. Monthly reports are then required from the mines thus licensed, showing the quantity of material mined and the condition of the dams.

Inspections.—Frequent inspections are made to ascertain whether dams are kept in repair, whether there is always ample storage capacity, and whether any mines are operating without licenses or without dams. For this purpose several employees of the Commission, often a Deputy United States Marshal, are constantly inspecting during the mining season, and members of the Commission visit mines from time to time whenever a special inspection is needed. The cost of these inspections, of advertising applications, and the expenses of keeping the records of the mines, their reports, etc., in the office, are all borne by the United States, the mine owners, of course, building and maintaining at their own expense the restraining works required.

The task of inspecting mines and keeping their records is no small one. The mining area covers about 450 miles in length and 40 miles in breadth, of rough and mountainous country, which must be covered several times each mining season by the inspectors. This is necessary to prevent illegal operations and to maintain control of the licensed mines. The number of applications for licenses now reaches 727, the greater proportion of which, however, are not now in force. A card index, with lists arranged according to counties and consecutive numbers, is necessary to keep accurate record of the conditions at each place.

Mining without licenses, or even dams, and the use of various tricks to avoid building and maintaining dams are practiced to a limited degree, but it is seldom that legal steps are necessary to stop this sort of illegal operation. Careful and frequent inspections, with occasional warnings, are usually all that is necessary to prevent all illegal mining of any importance.

FIG. 1.—A BRUSH DAM IN CALAVERAS COUNTY.

FIG. 2.—A LOG-CRIB DAM, WITH SPILLWAY, IN PLUMAS COUNTY.

In this way the debris from hydraulic mining has been regulated so that very little is now added to the old supply. These restrictive measures, unfortunately, have been too great to permit the resumption of hydraulic mining on the large scale formerly followed, but, on the other hand, they have permitted many mines to operate that otherwise would have had to remain idle.

There are now mined nearly 1 000 000 cu. yd. each year, which are stored in the cañons and ravines behind debris dams specially constructed for the purpose. Under the restrictions imposed, the lower rivers are now slowly improving. The Sacramento River is gradually lowering its low-water plane at Sacramento, and the effect of the tide is beginning to increase. Both of these indications are distinctly favorable.

Protection of the Navigable Rivers.—The other side of the Commission's duty is the study of the rivers of the Sacramento and San Joaquin systems, with a view to the preparation of plans for the treatment of these streams and their tributaries so that the injurious mining detritus may be kept out of the navigable rivers, and the streams restored to their former condition of navigability as far as may be needed.

The first step, after preventing the operation of mines where debris was not properly impounded, was the treatment of the larger tributaries to prevent the enormous quantities now in their beds from reaching the navigable streams.

In 1881 the State of California built a brush dam in the Yuba and one in the Bear River, with a view to impounding debris up to the crest of the dams. These dams were only a few feet high, and were constructed of brush, gravel and sand-bags. They were necessarily founded on the unstable gravel bed of the river. Neither dam withstood the first high water. Both streams have widely varying discharges, the Yuba ranging from about 300 cu. ft. per sec. in the summer and autumn to about 80 000 or 90 000 cu. ft. per sec. during floods. The Bear River varies from about 10 cu. ft. per sec. during the summer low water to an estimated flood discharge of about 15 000 cu. ft. per sec.

With these flood volumes it is evident that any dam to hold, if built on the treacherous gravel of the river bed, must be of considerable strength.

General Principles of Improvement.—In undertaking the formulation of a plan for these rivers, the Engineer Officers adopted a general line of work that was believed to be applicable. It consisted of three divisions:

1.—The construction of moderately high dams in the foothills where the rivers emerge into the valleys, and where the value of land is not great. These dams, being located where the slopes in the river-bed are comparatively high, were placed there with a view to sorting the heavy material that will stand on high slopes, from the fine material that will not, thus storing this heavy material where it will be impounded at least cost.

2.—Embankments and basins lower down the river, forming settling pools, where the slopes are flatter and where practically all the finer material can be deposited at all except high stages of the river by bringing the flowing water almost, if not entirely, to rest.

3.—Training walls in the remainder of the lower river, to confine the flow in selected channels, so that the large quantities of debris now in the river beds, outside these walls, should not be overflowed, and thus could lie undisturbed indefinitely.

After investigating various other plans, it was believed that the application of these principles would hold back all the debris that could be impounded, and would offer the best solution of the problem.

Yuba River.—It was decided to commence the work of restraining the debris of the Yuba River, as this stream has suffered more from mining detritus than any other in California, and is causing the most trouble in the navigable rivers. If the difficulties could be surmounted in this stream, the methods found best adapted to the purpose would be likely to be more easily applied to the other rivers.

After a study of several years, and after extended surveys in which numerous borings were made, a plan was adopted by the Commission and submitted to Congress in 1900. Much credit is due to Mr. Hubert Vischer, Assistant Engineer, whose work on this plan, under direction of the Commission, has been of much value. The plan has since been modified from time to time as the progress of the work rendered necessary. The estimated cost was $800 000, of which the State of California, under the provisions of the Caminetti Act, pays one-half. This project was adopted by Congress, and funds were appropriated therefor.

FIG. 1.—A LOG-CRIB DAM IN SIERRA COUNTY, SHOWING LARGE TIMBER USED.

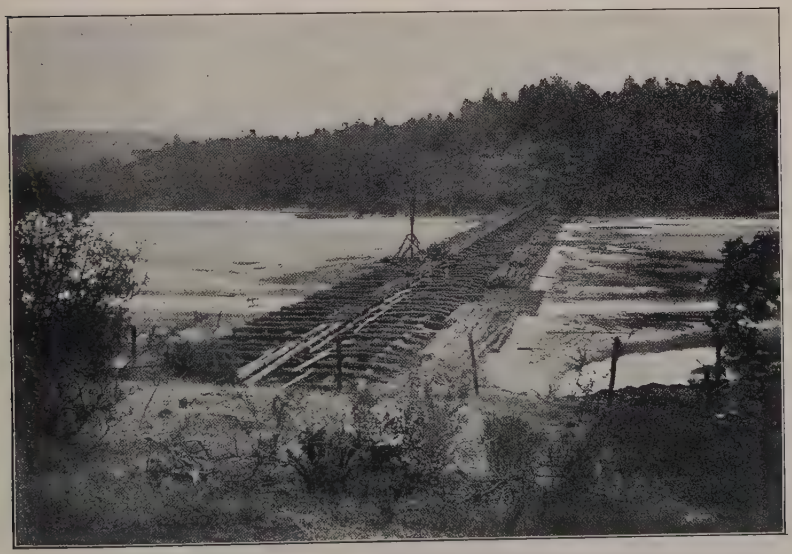


FIG. 2.—A BRUSH AND ROCK DAM ON THE YUBA RIVER, SUBSEQUENTLY DESTROYED BY HIGH WATER.

The project provided for:

1.—Barriers across the river just below Smartsville, to hold back the coarse detritus coming from the upper reaches.

2.—A cut at Daguerre Point, through which to divert the river at high stages, with embankments forming a settling basin for impounding fine material during the remainder of the year.

3.—Training walls, about 2 000 ft. apart, extending from Daguerre Point to the Feather River, to confine the flow to a selected channel.

Barriers.—The barriers are to be a system of weirs extending across the river, where the banks are high enough to afford large

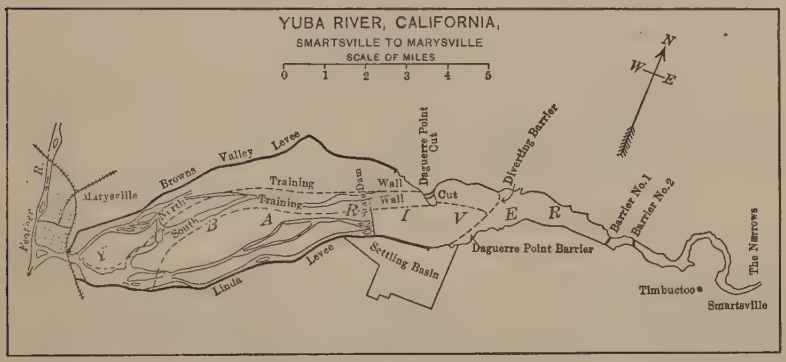


FIG. 3.

impounding capacity, the first located a few miles below Smartsville. This first barrier was the only one estimated for in the present project, but it was to be supplemented by others higher up the river as soon as it was filled, the others to be located and built when necessary.

A dam of brush, rock and gravel was first proposed, with a row of Wakefield sheet-piling 20 ft. deep to protect the toe. It was found impracticable to drive the sheet-piling on account of the coarse and heavy material of the river-bed, and, therefore, this type was abandoned after several hundred feet of it had been placed with much difficulty. This portion afterward washed out during a flood in the winter of 1903-04.

A modified brush barrier was then tried by the Commission. It was a "cob-house" construction of brush fascines, forming pens 5 ft.

square, with an elevation of 4 ft. above the river-bed. These pens were filled with heavy rock. An apron 20 ft. wide was made of a mattress of brush fascines fastened together with cables. This dam was destroyed by the first high water. The large amount of drift carried by this freshet broke apart the fascines, and the dam soon disintegrated.

The design next tried is much stronger. It is anchored to the river-bed with piles, and tied together longitudinally with two timber bulkheads. The first step of this dam, 6 ft. above the river-bed, is composed of rock fill held in place by concrete blocks weighing about 10 tons each, moulded in place over the rock fill, and connected with wire cables embedded in the concrete. Leakage is checked by the timber bulkheads. A broad apron, 20 ft. wide, with a 6-ft. lip, diminishes the scour at the toe, that otherwise might undermine the dam. A sloping up-stream face prevents damage from drift.

It is thus seen that most of the weaknesses of the previous dams have been remedied in the new type. This dam has passed successfully through its first high-water season without any sign of weakness, and is the first dam to withstand a single freshet in the lower Yuba River.

The dam is the first of a series of steps, of which the ultimate barrier will be composed, and consists of four rows of piles, the two upper intervals between the rows being 10 ft. and the interval between the third and fourth rows being 18 ft.

Piles are at 6-ft. centers in the uppermost row, at 12-ft. centers in the two middle rows, and at 3-ft. centers in the lowest row. Every 12 ft. the piles in a tier up and down stream are connected at their upper ends with 1-in. galvanized-wire cable. A timber bulkhead, 3 in. thick, is spiked to the up-stream and another to the down-stream row of piles, and is carried as deep as the water in the river would permit.

Between the first two rows of piles is placed a fill of rock which was brought up to a subgrade, so that when covered by the concrete blocks, 1½ ft. thick, the height of the barrier would be 6 ft. above the average level of the river-bed. Concrete blocks, about 10 ft. square and 1½ ft. thick, are built in place over all this fill, connecting, by a rollerway, with an apron 20 ft. wide resting on the river-bed below the dam.

FIG. 1.—BARRIER NO. 1, YUBA RIVER, SHOWING METHOD OF CONSTRUCTION.

FIG. 2.—FIRST STEP, BARRIER NO. 1, YUBA RIVER, COMPLETED, READY TO TURN THE RIVER OVER THE DAM.

The up-stream slope is protected by an inclined layer of large rock laid in Portland cement mortar. The concrete slabs of the top surface of the dam and those of the apron are separated from each other by tar-paper joints, and to prevent dislocation by the river currents are connected in the direction of the stream flow with 1-in. galvanized wire cables, 3 ft. apart, embedded in the concrete. The cables connecting the piles referred to above are also embedded in a narrow strip of concrete 18 in. square, which helps bind the heads of the piles together in each tier and separates the large blocks.

Excepting these narrow strips, the concrete slabs rest on the river bottom only, and are not supported on the piles. They are jointed so that they are free to move vertically, the cables acting as hinges.

This was designed to permit the concrete blocks to follow down any considerable scour under the apron, should it occur, and thus prevent any serious damage to the dam due to back-lash.

The weak place in all over-fall dams on poor foundations is, of course, the toe. The rollerway and apron are designed to help to protect the river-bed from excessive and dangerous velocities. In addition, an extension or lip of masonry 6 ft. wide is placed below the apron to carry the water away from the toe, and, if under-scoured, it will break up, fall into the hole and offer protection against further scour. This action occurred near the south bank for a short length during the high water of 1905, and was found to act effectively as planned. In addition, for about 600 ft., the dam at the south end, where the scour was believed to be strongest, was protected by large rock or rip-rap placed at random. This protection, which may be considered a flexible rip-rap apron, was added to during the next season, making it more than 20 ft. wide entirely across the river-bed.

The south end of the dam is joined to the bed-rock of the river bank, on which the south abutment is founded. At the north end, also, a concrete abutment was built, which was founded on piles, to act as a retaining wall for the earth embankment built later to connect the north end of the dam with the shore. The north shore is composed of compactly cemented gravel through which it was originally planned to have a spillway constructed to carry the river at all stages except flood. During construction, the entire river was

permitted to pass between this abutment and the north shore, before building the embankment, so that the first step was constructed entirely on the dry river-bed. When the time came to close this gap through which the river flowed, it was found to be an undertaking far greater than was anticipated, as there were about 1 200 to 1 300 cu. ft. per sec. flowing around the end of the dam, about three times the flow expected. This flow had to be lifted more than 8 ft. over the completed structure.

The first pile-work placed to close this gap failed on account of the pressure of the water and the serious scour which took place in the bottom while trying to close the opening. Later, more piles were placed in the opening, and, by the liberal use of brush and sand-bags, the gap was finally closed.

Much assistance was afforded by an auxiliary dam or levee in the river, ¼ mile long, hastily built to divert the flow over the dam and away from the cut. More than 20 000 sand bags were used in closing this gap, as no rock was available. As soon as closed, the gap was filled with an earth embankment.

The experience with the north bank near the end of the dam has decided the Commission to change the location of the permanent spillway from the north end to the south bank, where the cut will be in rock. This spillway will be 400 ft. wide on the lip and 4 ft. deep, and will be raised as the dam is raised. When completed, it will take a flow of 13 600 cu. ft. per sec. before the dam comes into use, so that the river will be carried around the south end of the dam at all times except for a short period during each year at flood stages. This will make the dam when completed practically permanent and easily maintained.

The first step of the permanent dam having proved so satisfactory, the Commission decided to build the second step during the following low-water season, and contracts were let in May, 1905. This work is now completed. The elevation of the second step is 8 ft., making the total height of the dam 14 ft. The general features of this second step are the same as those of the first step.

In the same way, it is expected to put a step about 8 ft. high on the dam as often as conditions warrant. In this way the ultimate height of the dam (36 ft.) will be reached by successive steps. This method was found advisable, as the amount of work possible in the

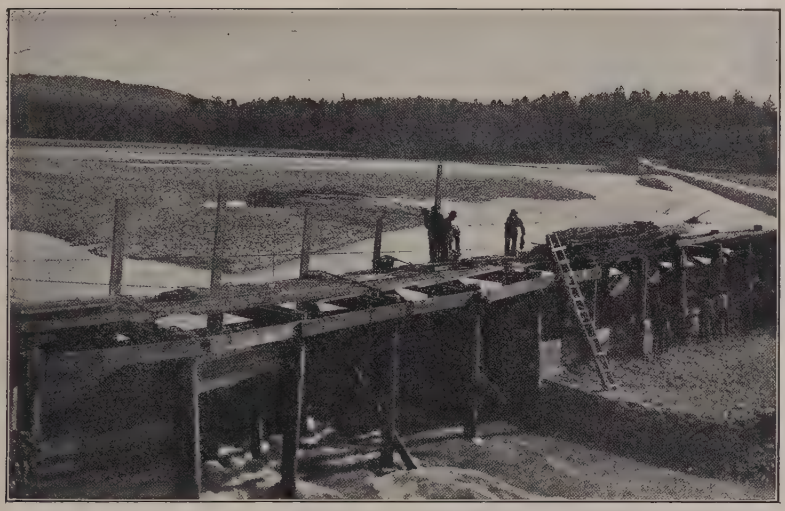


FIG. 1.—FIRST ATTEMPT TO DIVERT RIVER FLOW OVER THE BARRIER. UNSUCCESSFUL.

FIG. 2.—PORTION OF RIVER CHANNEL USED DURING CONSTRUCTION, CLOSED, PREPARATORY TO FILLING WITH EARTH EMBANKMENT.

river-bed during the low-water season is limited, and time must be given to the river to fill each step with gravel.

The first step, already built, has been filled with gravel to its crest since the first heavy freshet, and gravel as large as pigeons' eggs has been rolled over the top of the dam for several months. This has given rise to considerable wear of the concrete, in some places more than 2 in. deep, during the winter.

It is thus plainly seen that the dam as a whole is a gravel-fill dam sluiced into place by the river itself, the down-stream slope of which is composed of a layer of concrete blocks having a general inclination of about 1 vertical to $3\frac{1}{2}$ horizontal. The concrete overlies a rock fill held in place by a framework of anchor piles and timber bulkheads.

The concrete for this work was made of gravel taken from the river-bed. It was screened and measured in order to get the proportions of the various sizes decided upon. The materials were unusually clean, white quartz sand, gravel and small cobbles. Cobbles were selected that would pass a 2-in. ring and be held by a 1-in. ring, gravel that would pass a 1-in. ring and be held by a $\frac{1}{4}$-in. ring, and sand passing a $\frac{1}{4}$-in. opening was used, care being taken to select sand from places in the bed where but little fine material was found. A number of experiments for voids were made, all showing that the interstices in each size varied from 32 to 35 per cent. The average was about 33%, which was assumed to be the average for each size. The proportions used were: 1 cement, 2 sand, 2 gravel, and 4 small cobbles, for the first season's work, and an additional volume of gravel was added for the second season's contract, making the proportions 1 : 2 : 3 : 4. The concrete was hand-mixed in the first year and machine-mixed in the second year, effecting a great saving in cost to the contractors for the later work. A local brand of Portland cement was used. The resulting concrete was thoroughly satisfactory for the purpose. A wearing surface of 1 to 3 mortar, 2 in. thick, was placed over the concrete in the first year, but this wore through in places with the abrasion caused by the gravel which passed over the dam. It was found that the quartz aggregate of the concrete was the best to resist the abrasion, and the surfacing will probably be omitted in the future.

Cost.—The first piling was purchased under contract, but driven

by the United States, because it was not believed advisable to contract for this work at first, as the risk was considerable, and it was freely predicted by some engineers that no piles could be driven in the bed of the Yuba River at that place. Excepting delay due to overturning and wrecking the driver twice by high water, this work was carried to completion without notable difficulties, 888 piles being driven with an ordinary land driver equipped with a 20-h. p. Lidgerwood engine and a 3 500-lb. hammer working in 45-ft. gins. The driving was difficult, each pile requiring from 150 to 250 blows. The piles cost from 19 to 23 cents per ft. delivered in Marysville, and 10 cents per ft. to haul to the site of the work, 17 miles further. This made each pile cost about $12. The driving cost $5.02 per pile, making the cost of each pile in place about $17. Including experiments with water-jet and accidents, the cost was $19.74. This price was considered high at the time, but the work under the contractor for the piling of the second step, in the spring of 1905, has exceeded this price. Under contract, the new piling for the second step has cost the Commission 31½ cents per ft. per pile delivered at the site of the work, making each pile cost $12.60. The contract price for driving is $7.60, making each pile under contract cost $20.20 in place.

Portland cement costs $2.785 per bbl. delivered at the site of the work. It was furnished by the contractor during the first season, but by the United States during the second.

The cost of the work on the first step, including the abutment, is shown in Table 1, the prices including placing:

TABLE 1.

Excavation	6 470	cu. yd.	at	$0.30	$1 941.00
Lumber	61 655	ft., B. M.,	at	$40.00	2 466.20
Loose brush	95	cords	"	3.50	332.50
Large rock	2 411.75	tons	"	2.00	4 823.50
Large rock in cement	1 621.307	cu. yd.	"	7.00	11 349.15
Small rock fill	4 251	tons	"	1.00	4 251.00
Cable	30 300	lin. ft.	"	0.18	5 454.00
Concrete	3 754.2	cu. yd.	"	7.50	28 156.50
Extra work					1 300.10
Total					$60 073.95

FIG. 1.—PILING COMPLETED FOR SECOND STEP OF BARRIER.

FIG. 2.—SECOND STEP OF BARRIER NEARLY COMPLETED.

At the prices given in Table 1 only a small profit was made by the contractor, according to memoranda kept by the engineers during the progress of the work.

The entire dam, including piles, but not the north abutment and embankment, cost, in round numbers, $78 613, or about $63 per lin. ft.

The construction of the first step of the barrier was successful, and its behavior was satisfactory during the high water of the past winter. This enabled the Commission to take up the construction of the second step of 8 ft. higher during the following low-water season. Accordingly, a contract was let in February, 1905, for delivering 400 piles at the site of the work at 31½ cents per lin. ft., this cost being made up of about 21½ cents on the cars at Marysville and 10 cents for hauling them 17 miles to the barrier. A separate contract was let in February for driving the piles at $7.60 per pile. All piles, 335 in number, were driven by June 15th, 1905.

The piles were 40 ft. long, driven to 30 ft. penetration in the river-bed, leaving 10 ft. standing for the step, including a small cut-off.

A contract was let for the main part of the second step in April, and by June 15th, 1905, work was commenced. The main features of the second step are similar to those of the first step, and are shown on Plate II. The work was carried on without interruption from high water, as had happened twice in constructing the first step, and by November 25th, 1905, had been completed. In carrying on this work, the entire river flow, amounting in the late summer to only about 300 cu. ft. per sec., was thrown into a selected channel at the south end of the barrier, which was protected from scour by brush mattress loaded with rock. This enabled the entire dam, of about 1 250 ft. length, except about 120 ft. at the south end, to be constructed in the dry bed of the river. Upon the completion of the main work, a double row of piles at this south end, specially provided for this purpose, was bulkheaded and then filled with rock. This formed a coffer-dam around the unfinished end of the dam, which thus turned the water over the main dam, permitting the small unfinished section to be built without interference.

A flexible apron of rip-rap rock, 20 ft. wide, was laid across the

river-bed below the toe of the first step to protect the bottom from scour. Rocks weighing from 500 lb. to 2 tons were selected for this portion of the dam, 5 472 tons being used. This will be added to, during each low-water season, as needed. A south abutment was commenced, of which the only portion built was that necessary to provide for foundations for future work. This abutment is founded on rock, and will form a part of the spillway to be built later at this end.

The contract prices and the quantities used for this second step are given in Table 2.

TABLE 2.

Excavation	2 985 cu. yd. at $0.75 per cu. yd.	$2 238.75
Embankment	11 520 " " 0.30 " "	3 456.00
Bulkhead lumber	61 000 ft. B. M. at $25.00 per M.	1 525.00
Rock fill	13 223 tons at $0.875 per ton	11 570.13
Large rock	5 409 " " 2.00 " "	10 818.00
Concrete	4 156 cu. yd. at $4.25 per cu. yd.	17 663.00
Wire cable	15 600 lin. ft " 0.15 " lin. ft.	2 340.00
Brush mattress	560 sq. yd. " 0.70 " sq. yd.	392.00
Sand bags	1 600 at $0.07 each	112.00
Total		$50 114.88

Records kept during progress of the work show that the contractors made no profits at these figures.

To the total in Table 2 must be added the cost of cement, 4 100 bbl. at $2.785 per bbl., the cost of piles, $6 556, furnished under other contracts, and the cost of superintendence and inspection, amounting to $2 264.

The cost of the second step, not including the abutments or the rip-rap apron below the dam, was $37 575, or $30.06 per lin. ft.

The rip-rap protection below the apron, placed during 1905, consists of 5 472 tons of large rock, costing in all $13 604, or $11.83 per lin. ft.

The cost of the entire work of construction of the first two steps, but not including the end abutments, has been $129 792, or $103.83 per lin. ft.

Daguerre Point Section.—The plans for the treatment of the intermediate section of the Yuba River involve the construction of high embankments of river gravel extending entirely across the

FIG. 1.—TOP VIEW OF BARRIER; SECOND STEP NEARLY COMPLETED; RIVER PASSING AROUND FARTHER END.

FIG. 2.—SECOND STEP OF BARRIER COMPLETED. DEBRIS ON STEPS WILL BE REMOVED BY FIRST HIGH WATER.

river in a V-shape, with the apex up stream, the down-stream ends connecting, one with Daguerre Point on the north, and the other with a high knoll on the south bank. These barriers are to be so high that they will never be over-topped.

A diverting barrier connects the apex of the V with the north shore, diverting to the south all water below the elevation of its crest. Through Daguerre Point is being cut a channel, 600 ft. wide and 25 ft. deep, through which will pass all the river flow at high stages.

On the south bank, regulating works will admit all the water turned by the diverting barrier, up to a limit of about 3 000 cu. ft. per sec., passing it into a natural depression of about 2 sq. miles area, adjoining the river on the south. These works will exclude all flow greater than what is considered safe, compelling the excess to pass to the north over the diverting barrier and through the cut. This plan is simply taking advantage of the natural regimen of the river.

All rivers have their sections of active erosion, usually where slopes and velocities are highest; their sections of transportation, where slopes and velocities are sufficient to carry sediment, but not to scour; sections of sedimentation, where the reduced velocities permit the sediment to fall and form deposits; and their sections of discharge.

The construction of the embankments and the use of a large settling basin only increase the natural area of the section of sedimentation. The river is passed into an area where the velocities are largely checked, although not entirely overcome, so that practically all their load of sediment is dropped. The water passing back into the river at the lower end of the basin will be practically clear. At stages of the river, occurring for a short time only during each year, when the flow of water through the settling basin would be more than 3 000 cu. ft. per sec., and more than could be settled, the regulating devices will exclude the excess, causing it to pass into the cut. As the river is below this limit for by far the greater part of the year, this will happen only at intervals, and then at such high stages that the velocities are thought to be sufficient to carry whatever fine sediment is in suspension into the Feather and Sacramento Rivers, which, being at flood at the same time, will

carry the sediment into the tidal currents of the bay and thence into the ocean. As the periods of such high water are short, the damage to the navigable rivers, if any should result, will be limited, and will in any case be less than the river can repair with its own scouring action.

The work of excavating the cut at Daguerre Point has been in progress for more than two years. About 650 000 cu. yd. have already been excavated, and the work is now nearing completion.

A steam shovel having a 1½-cu. yd. dipper is used, the excavated material being removed, by two trains of about 10 cars each, to the dump at the north side of the entrance. An average of 30 000 cu. yd. per month is required under the contract, which will be completed by the close of the calendar year. The contract price of excavation is 23½ cents per cu. yd. of earth and 90 cents per cu. yd. of rock removed. The total cost of this cut will be somewhat more than $160 000.

Lip.—In order to prevent unusual scour through this cut, which must take nearly all the maximum flow of the Yuba River, amounting in floods to as much as 90 000 cu. ft. per sec., and perhaps more, it was necessary to construct an inlet wall or sill at the entrance. This is of masonry, and is sufficient to hold the bottom and banks in its vicinity during this extreme flow. Accordingly, contracts have been let for a reinforced concrete sill with a reinforced concrete abutment at the north end and with a rock face for an abutment at the south end. This work is now under construction. The length will be about 700 ft.

The estimated cost of the work is given in Table 3, the prices being those of the contracts now in force:

TABLE 3.

Excavation	2 200	cu. yd.	earth,	at	$1.00	per yd.	$2 200
"	200	"	rock,	"	1.50	"	300
Back-fill	140	"		"	1.00	"	140
Embankment	300	"		"	0.75	"	225
Concrete	2 340	"		"	5.00	"	11 700
Plain steel bars	30 100	lb.		"	0.06	per lb.	1 806
Twisted steel bars	8 300	"		"	0.07	"	581
Excavation at entrance.	32 000	cu. yd.	earth,	"	0.45	per yd.	14 400
	1 000	"	rock,	"	0.45	"	450
Total							$31 802

Cement is furnished by the Commission, and costs $2.25 per bbl. in Marysville and 41 cents per bbl. for hauling 11 miles to the site of the work.

The average cost, excluding abutments and the excavation of material in the entrance, will be approximately $35.40 per lin. ft. of length of the wall.

It was originally intended that the United States should build the high embankments extending across this part of the river, but before operations could be begun it was found that a gold dredging company had secured extensive mining rights in the vicinity, and that this promised to conflict decidedly with the plans of the Government. It was also learned that the company, in the exercise of its rights, proposed to dredge for gold in the river-bed just where the original settling basin had been located. After considerable controversy, an amicable agreement was reached by which the Commission was enabled to obtain a larger settling basin on the south side of the river in exchange for the location first adopted, certain deeds to property essential to the project were secured, and the dredging company also agreed to build the necessary embankments free of cost to the United States. The area mined was to be returned to the United States when it had been completely dredged, for such use as could be made of it. The embankments are of considerable value to the dredging company, as they protect its operations during high water, but they are built along lines proposed by the United States, and save the cost of construction to the United States. The total length of embankment will be about 2½ miles. About ½ mile has been built already with only two dredges. Six more dredges are under construction, and will be placed on this embankment work as soon as completed. The embankments now being built are about 30 ft. high and from 200 to 300 ft. wide on the base. They will not be over-topped. These embankments will be completed, it is expected, by the close of 1906.

Training Walls.—Below Daguerre Point the debris in the river-bed is from 7 to 25 ft. deep, increasing from Marysville as one proceeds up stream, and forming a body about 2 miles wide, on an average, and more than 9 miles long. The project contemplates confining the river to a selected channel and leaving the remainder of the old bed undisturbed. Portions of the old bed may be used for

settling basins, but the unrestricted movement of the river over this abnormal width of bed is to be prevented. This will be effected by dikes at proper distances apart, which will contain the maximum flow. This will undoubtedly result in some scour in the selected channel until the equilibrium of the forces at work has been more nearly reached. This action, however, will all be in the direction of greater security from floods. Already, agreements have been made with two gold dredging companies to build 2 miles of these walls, lying on the south side of the channel below Daguerre Point, free of cost to the United States. The north wall for a corresponding length of 2 miles is now being built under contract by the Commission at a cost of 12.4 cents per cu. yd. of material in place. The total quantity in this north wall will be about 200 000 cu. yd. The distance between these training walls at the upper end is 2 000 ft. The distance apart will be adapted to the slope and the height of the walls in the lower reaches.

Other Rivers.—During the summer of 1905 a survey party was kept in the field surveying Bear River, with a view to its treatment along similar lines. The Bear has been surveyed from its mouth in the Feather up to the mouth of Greenhorn Creek, a distance of 32 miles, sites for dams have been examined, and a plan for work is now being formulated.

The American River will next be studied, and a plan for its treatment prepared.

When these other rivers have been remedied, the worst offenders will be rendered harmless, and the Commission can then turn its attention to the other problems of navigation and flood control in the Sacramento Valley.

DISCUSSION.

H. H. WADSWORTH, M. AM. SOC. C. E. (by letter).—Since the publication of Captain Harts' paper some additional data have been obtained relative to the hydraulics of the Yuba and Bear Rivers, and the movement of debris by them. Mr. Wadsworth.

The construction of Barrier No. 1 has rendered possible a fairly close estimate of the quantity of water flowing in the Yuba River at various stages. A comparison of the cross-section of this dam with those of several forms of weirs, the coefficients of which in the usual weir formulas have been determined by several experimenters, gives an approximate value for its coefficient. During the high water of January, 1906, the depth of water on the crest of the dam was 5.3 ft. at the north end and 6.0 ft. at the south end. These depths indicate a flow of about 70 000 cu. ft. per sec., a determination which has been roughly checked by the United States Geological Survey at a measured cross-section near Smartsville.

During the flood of February, 1904, said to be the greatest for many years, the gauge height at The Narrows, about 6 miles above the barrier, is reported to have been 4 ft. higher than during the flood of January, 1906. This extra depth means a discharge well up toward the 90 000 cu. ft. per sec., stated by the author to be the probable maximum.

On the Bear River no recent flood discharge measurements have been made, but an extension of velocity and discharge curves, plotted from observations taken by the U. S. Geological Survey for several gauge heights, up to the maximum gauge heights observed, in connection with measured cross-sections, show that the assumed maximum flow for the Bear was nearly or quite reached during the flood of January, 1906.

The recent survey of the Bear River shows that conditions there have reached a state of comparative equilibrium.

The form of the banks at the confluence of the Bear and Feather Rivers indicates that very little material is deposited in the latter by the former, and up stream, near Wheatland, the fact that alfalfa and hop fields are being cultivated between the levee lines shows that no great progressive damage is now being done from year to year.

On the Yuba, however, in spite of the fact that hydraulic mining on a large scale was stopped more than twenty years ago, there is still an extensive movement of the mining debris, as pointed out by the author, and this is likely to continue for many years. The great bulk of the material is moved during the few days of extreme high water each season. During the flood of January, 1906, the pool formed by the second step of the barrier was filled to the

Mr. Wadsworth.

crest of the dam, at the center of its length, and the new deposit, extending up stream about 1½ miles, measured more than 920 000 cu. yd. The continuance of a fairly high stage of the river since the flood mentioned has filled out the corners and increased the deposit to more than 1 000 000 cu. yd.

The filling in behind the first and second steps of the dam in such a short time is roughly confirmatory of the results of the survey, which showed a fill of 15 000 000 cu. yd. of material in the stretch of river bottom between Marysville and Barrier No. 1 in the five years between 1899 and 1904.

DIAGRAM SHOWING THE VARIATION OF FLOW IN THE YUBA RIVER DURING THE FLOOD OF JANUARY 1906.

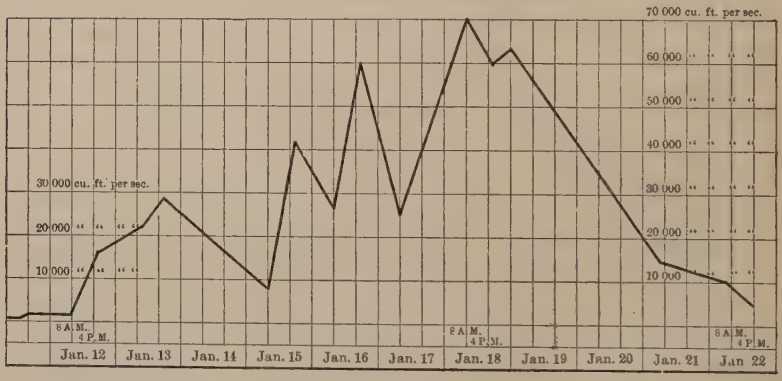


FIG. 4.

The diagram, Fig. 4, shows approximately the variation in the flow of the Yuba during a portion of January. This shows graphically the flashy character of the floods. The total discharge between 8 A. M. on January 12th and 4 P. M. on January 22d was approximately 1 000 000 000 cu. yd. of water. During this period was deposited the great bulk of the fill above the dam, which, from the foregoing figures, amounted to less than one-tenth of 1% of the mass of water by which it was transported. In addition to the material pushed along the bottom by the flood wave, a large quantity, of course, was carried over the dam in suspension. This may easily have amounted to one-third of the quantity deposited above the dam.

During the present season a spillway of sufficient capacity to carry the entire flow of the river for six months of the year will be built. As succeeding steps are added to the dam the spillway will be increased until, when the final height of the dam has been reached, its capacity will be such that water will flow over the crest of the dam only during extremely high water.

Besides the relief which this spillway will afford to the dam, its value as an accessory in the construction of succeeding steps will be very great, as the difficulties of turning the water and making the closures will be avoided. Mr. Wadsworth.

FRANKLIN RIFFLE, M. AM. SOC. C. E. (by letter).—The author has properly prefaced his excellent description of the methods used to control hydraulic mining in California by a brief historical review of this important industry. This feature adds materially to the value and interest of the paper. The development of hydraulic mining gave rise to many unique engineering problems which were boldly attacked and admirably solved by California engineers, whose achievements are but slightly known to civil engineers outside of that State. It is much to be regretted that an industry which has contributed so largely to the material wealth, not only of California, but of the entire country—which has placed in the hands of engineers hydraulic data of incalculable value—and is so distinctively an American enterprise—is not mentioned in the index of papers published in the *Transactions* of this Society. It is greatly to be hoped that some California member, familiar with the history of hydraulic mining, will rise to the occasion, and contribute a suitable paper on this subject before many important details of the early stages of the industry are lost to the engineering profession. Mr. Riffle.

The evolution, from small volumes of water, low pressures, canvas pipe, and crude, improvised nozzles, to large volumes of water, high pressures, heavy double-riveted pipe of large diameter, and the hydraulic monitor or giant, took place in a surprisingly brief period, and, when taken in connection with the construction of ditches, flumes, tunnels, dams and reservoirs, often under unprecedented conditions, marks an important epoch in engineering history which deserves to be fittingly commemorated in the annals of America's leading society of civil engineers. It is well to remember that many of the great things accomplished by the engineers of to-day are to a very great extent the direct result of the experiments, mistakes, failures and successes of the engineers of yesterday. And so in California the engineers who were engaged in developing the industry of hydraulic mining more than a quarter of a century ago, through their failures and successes, have prepared the way for many of the spectacular achievements of the hydraulic and electrical engineers of the present day.

The author's statement that "the industry of hydraulic mining was finally completely vanquished," although evidently intended to apply to the region drained by the Sacramento and San Joaquin Rivers, may be misleading to those who are not residents of California. Hydraulic mining in California has never been prohibited by statute, although the decree of the United States Circuit Court

Mr. Riffle.

which perpetually enjoined and restrained the North Bloomfield Mining Company from discharging mining debris into the Yuba River or any of its tributaries, had the effect of closing all hydraulic mines, the operation of which clearly caused injury to navigation or private property. In the northern part of the State, however, in the region drained by the Klamath River, hydraulic mining has been prosecuted without any interference whatever. Section 1424 of the Civil Code of California provides that:

"The business of hydraulic mining may be carried on within the State of California wherever and whenever the same may be carried on without material injury to the navigable streams or the lands adjacent thereto."

These conditions exist in the territory drained by the Klamath River. This, the second river in size in the State, is a typical mountain stream. It has a rapid, torrential current, and flows through a succession of deep gorges which extend almost to its mouth. There are practically no agricultural lands along its banks, and it is hopelessly unnavigable. High benches of auriferous gravel aggregating thousands of acres lie along its banks and those of its tributaries. Hydraulic mining operations have been conducted in this locality for many years with satisfactory results, the gravel yielding from $5 000 to $25 000 per acre. Although most of the mines are small, their aggregate yearly production of gold is very considerable. A few mines, however, operate on a large scale. The La Grange Mine, in Trinity County, for instance, has one of the largest hydraulic plants in the world. With high banks (the maximum height being 300 ft.) and 9 000 miner's in. of water available, it has been possible to hydraulic from 10 000 to 12 000 cu. yd. of gravel per day. To convey this large volume of water from the source of supply to the mine it was necessary to construct 16 miles of flumes, ditches, pipes and tunnels. This conduit contains a tunnel 1½ miles long, and a 30-in. riveted siphon nearly 1 mile in length operating under a pressure of 450 lb. per sq. in. at its lowest point.

Without going further into detail, it may be concluded that hydraulic mining promises to be an important industry in California for many years—especially in these localities which are more fortunately situated than the region described by the author.

Mr. Galloway.

J. D. Galloway, M. Am. Soc. C. E. (by letter).—The engineers in charge of the construction of the Yuba barrier are to be congratulated on the success of their effort to build even a low dam across the Yuba River. Since the paper was written the river has been in flood, and, as no damage to the barrier has been reported, it may be considered that the second step in the dam will stand.

The writer is familiar with the Yuba River, and offers the following comment upon the subject of the paper. It does not seem,

from the author's statements that any reliable estimate could be made of the quantity of debris now in the channels of the various forks of the river. Hence the ability of works, as designed, to control the situation, is open to question. According to the author, about 1 000 000 cu. yd. of earth per year are being mined, and this is restrained by log and brush dams. It is the writer's opinion that, in time, this will result in renewing the problem of the lower river. The brush and log dams will certainly decay, and, if upon any considerable stream, the debris will then be carried to the lower reaches and deposited. Owing to the fact that most of the tributaries are nearly or quite dry for a period every year, the decay of the dams will be rapid. Mr. Galloway.

The head dam of the Bay Counties Power Company, 8 miles above Colgate, was formerly a wood-crib dam about 40 ft. high. It failed two years ago, after a service of some thirteen years, and was replaced by a stone dam. The writer examined the timber of this dam and found that, although it was in fair condition, it showed evidences of decay. Before failure the river was filled to the dam crest with debris, and of course, this went out with the dam. The plan of this dam was very much like the plan shown by the author, and its failure is cited as an evidence that the dams built under the permits of the Debris Commission will fail in time, and in most cases the impounded material will be washed down the river. It is not quite as bad, however, as turning the debris directly into the stream, but, if the practice is continued, the condition of affairs will in time be nearly as bad as ever.

Referring to the impounding of debris by the barrier, the author does not state the cubic contents of the basin above the dam, nor of the settling basins below. The quantity of debris brought down by the river is given as about 3 000 000 cu. yd. per year, but, without knowing the storage capacity of the basins, the ability of the works constructed to control the situation is not evident. A statement is made that after the first step was completed in 1904, the first freshet filled the basin to the crest of the dam.

The second step of the dam of course added a greater cube to the basin, and the author probably now knows what effect the late floods have had upon it.

Referring to the dam, the writer believes that the addition of two more steps, making it 36 ft. high, will be a serious menace to the entire structure. While part of the flood will be carried by the spillway, a relatively small quantity of water may do considerable damage to the toe. Water passing the series of rollways does not lose much of its energy, most of it being expended in the pool at the toe.

The training walls from Daguerre Point to Marysville seem to

Mr. Galloway. the writer to be the best part of the scheme. Their value lies, not so much in lands reclaimed, as in preventing the river from washing the entire deposit of debris into the Sacramento River below.

The foregoing is not to be considered as a criticism on the design of the dam. It has been built on the shifting sands of a violent river, and the engineers are to be congratulated upon the success of the work. It is believed, however, that the building of temporary dams in the mountains to restrain debris is bad policy. Gold is not wealth, and the State would be better off if hydraulic mining had never been invented.

It is possible that the author has at hand the capacities of the basin above the barrier, intended to restrain the material of larger size, and of the settling basin for the finer material, in which case, an estimate of their ability to impound the debris could be made. Without this, it would seem that the project is open to question, as suggested above.

Mr. Richards. H. De C. Richards, M. Am. Inst. M. E. (by letter).—Mr. Hart's very able, comprehensive and highly educational paper refers to the protection for navigable streams developed on account of the great damage done by dumping mining debris into the tributaries of those streams, but no reference is made to the debris from slides, which is steadily increasing and, in some sections, is much more serious than the mine dumpage.

The "Caminetti Law," so-called, has resulted less expensively in the long run for the miners than the fighting of many debris suits—some honest and many blackmailing.

The writer will describe briefly the conditions on the non-navigable streams of Northwestern California.

Several years' experience in hydraulicing gravels in Humboldt and Siskiyou Counties impels the writer to state that there are four counties in California where hydraulic mining can be done without injury to navigation or agricultural interests. The counties of Del Norte, Siskiyou, Trinity and Humboldt (north of Eureka) have an area of some 8 000 000 acres, or about 12 000 sq. miles.

The Klamath and Trinity Rivers, and the tributaries thereto having workable auriferous gravels within reasonable dumping distance, cover a length of about 600 miles. The lower 60 miles of the Klamath River has a fall of about 490 ft.

Placer mining began in that section about fifty years ago, and seems to have increased steadily in number of openings until some ten years ago, when many of the larger workings had reached the hill rock. The small "one-man workings" are innumerable; a few large mines are still operating on the Klamath, Trinity, Salmon and Scott Rivers, all of which, with the many smaller streams, flow

to the Pacific through the Klamath. The lower 45 miles of this river has no large mines; the gravels are on the upper benches, some as high as 2 000 ft. above the river, with no water available for mining. Mr. Richards.

The mine now being operated by the writer at Orleans has averaged a clearing of about 1 acre of bed-rock per month with a full flume (5 by 3 ft., with a gradient of 10 ft. per mile) under a head of about 250 ft., and with banks from 50 to 80 ft. high. New workings, just opened, give 173 lb. static pressure on the gauge, that is, 402 ft. head. The same gauge indicates 120 lb. on the rear casting of a No. 4 giant, with a 6.5-in. nozzle, and with an overflowing penstock. There are several larger workings on that river, and the season is of about the same length, from 8 to 10 months—usually 8 months.

The whole country "slides easily." During the wet season, really phenomenal slides occur, stripping precipitous patches of bed-rock sometimes an acre or more in area and having depths of more than 100 ft. In one instance a length of about 1 500 ft., and more than 100 ft. thick, slipped into the Klamath River, completely damming it for some hours. Of course, much damage was done a few hours later when it broke through. In the writer's opinion, the mining has not in the past and does not now put as much debris in the rivers as the slides in wet years.

Carefully kept rain gauges have recorded as follows:

1900-01	51.54 in.
1901-02	51.68 "
1902-03	59.30 "
1903-04	82.10 "
1904-05	44.27 "

In spite of these conditions, the rivers have not filled up, and, in fact, the Klamath and Trinity Rivers are so swift that they almost reach the normal scoured condition at any heavy flood season. Bars form and disintegrate from season to season, according to the stage of the water. The formation of a bar at any point of course indicates an increased jettying beyond.

Indian dugouts are the only craft the rivers float, and, aside from casual blackmail suits, assisted by legal gentlemen on contingent fees, there is little to fear from legal restrictions in that section. The Honorable John F. Davis says that in these river basins the only foe with which the industry has to contend is the occasional blackmailer. The writer might add that the ranching which might be affected by the mining is ridiculously small, and that all the ranching land is nearly always more valuable for mining.

Mr. Richards. Many excellent quartz leads have been opened in that section, and a large number are mines in operation. Prospecting is more than encouraging in some of the newer districts, but is helped by the placer mining, and there are no complaints.

The following is from the Government Report on gold and silver production in 1904:

"Hydraulic mining property is in small demand in that area (drainage basin, San Joaquin and Sacramento Rivers), and few new mines are opened. Numbers have had their licenses withdrawn for violation of the rules of the commission granting them. Some changes in the laws will have to be made if hydraulic mining is ever again to flourish in this section.

"It is proper to note, and should be borne in mind by those interested, that these conditions exist only in the drainage area referred to. In all other parts of the State there are no restrictions on hydraulic mining. The extensive hydraulic operations in Siskiyou and Trinity Counties, and the smaller ones, in Humboldt and Del Norte Counties have always continued without interference by the people or the laws; and it is for this reason that Trinity and Siskiyou Counties now lead all others in this branch of mining. The drainage of all the streams in that area is ultimately into the Klamath River, which is a non-navigable stream, and empties directly into the Pacific Ocean, where all the debris finally disappears without possibility of damage to any one.

"Each mine may therefore dump its debris at convenience, and the spring freshets carry it out, leaving a place for another dump the following year. Naturally, under these conditions (which formerly prevailed in the area of the State now under restriction), no impounding basins for tailings are necessary, nor is there any Federal supervision whatever. The quantity of gravel to be washed is therefore limited only by the quantity of water which can be brought to bear under pressure against the banks of gravel."

Mr. Kieffer. Stephen E. Kieffer, Assoc. M. Am. Soc. C. E. (by letter).—The author, in his analysis of the physical and industrial conditions leading to the cessation of hydraulic mining in the tributary water-shed of the Sacramento River, and the creation of the California Debris Commission, has clearly given the underlying reasons for the existence of a problem, with the proposed method of its solution, that has proved too large for the State of California to cope with unaided, and too complex and bitter, politically and socially, to be dealt with in direct control by other than the Federal Government.

The bitterness engendered by the fight of a quarter century ago is a thing almost unknown to later comers in the State, especially if residence or business has not placed them in direct touch with the territory affected.

To-day, however, the feeling of animosity between the people of the valley and the people of the hills remains much the same as

ever; the miners ever hope to mine and the ranchers in the valley jealously watch to see that the danger does not threaten. Mr. Kieffer.

As evidence of this fact may be cited a meeting of the ranchers, in the fall of 1904, at Marysville, called to consider the question of proceeding against the gold dredges at work along the Feather River at Oroville, and on the Yuba at Daguerre Point—the two latter dredges being those referred to by the author as building the lower diverting barrier across the Yuba.

Floods during the previous season had covered with sand large areas along the Feather River, and this was attributed to the disturbance of the gravel deposits in the rivers by the dredges, despite the fact that, of the forty dredges operating, only three were directly in the river channels and handling recent debris deposits.

When it was pointed out that the effect of the dredges was to cover the unstable surface sands and fine gravel with 30 ft. of coarse cobbles, thereby forever restraining the fine deposits from further movement in the stream, the agitation came to a sudden end.

At about the same time an attack was made upon the dredges of the American River field, the press of Sacramento claiming that the city pipes were filling with sand as a result of the dredges excavating gravel at Folsom, 22 miles distant.

These dredges were operating at a distance of $\frac{1}{4}$ mile from the channel of the American River, from 40 to 60 ft. in elevation above it, and tailed back into the river from $\frac{1}{2}$ to 1 cu. ft. of water per sec. to each boat.

These cases serve to indicate how vigilant a watch is kept upon gravel mining in any form.

No other comment is needed upon the damage resulting to the valley lands from former hydraulic mining. While this vigilance guarantees immunity for the future, the writer feels that excessive alarm is unnecessary. To all intents and purposes hydraulic mining in the Sacramento water-shed is a thing of the past.

The author estimates that 1 000 000 cu. yd. of gravel are excavated per year at the present time. Taking his estimate of the yardage in the streams as follows:

Lower Yuba	333 000 000 cu. yd.
Bear	66 000 000 " "
Sacramento	108 000 000 " "
Other tributaries (assumed by writer)	100 000 000 " "

There is a total of 607 000 000 cu. yd. of gravel remaining in the streams.

Mr. Kieffer. If to this is added the *pro rata* of material estimated by the State Engineer in 1880 as being carried to Suisun and San Francisco Bays in suspension, the grand total is 760 000 000 cu. yd. of gravel mined and dumped into the streams in a period of about 25 years. No doubt the true volume is greatly in excess of this.

Upon this basis, it would take 760 years, at the present rate of hydraulicing, to cause a trouble of the magnitude confronting the Commission to-day, granting that all the gravel was allowed to go into the streams.

Comparatively few gravel deposits of the Sierras suitable for hydraulicing on any large scale have favorable sites for the storage of debris.

The necessarily small scale of the operations, combined with the added cost per cubic yard for restraining works, confines operations to the better paying or more favorably located gravels; hence, as long as the present regulations are in force, the field can never be greatly enlarged.

Granting that it is an open question as to whether or not the type of restraining dams specified will prove a permanent barrier to the impounded debris, the writer cannot see that the occasional failure of a dam, in the course of time, can have any significant effect upon the problem as a whole, provided the general restrictive regulations of the present are enforced.

As far as possible, hydraulic mining should be encouraged. This the Debris Commission has apparently endeavored to do, but the results at best can be no more than a virtual wiping out of the industry in the Sacramento water-shed, thereby surely solving this phase of the problem.

The more momentous problem of what to do with the millions of cubic yards of debris already in the streams, however, is not as easily solved by restrictive measures. The Yuba project, so fully outlined by the author, cannot fail to impress one with the magnitude of the undertaking.

Although the Yuba is the worst offender, it is adapted to the plan of holding back the debris better than any other river of which the writer knows. Probably physical conditions could not be found elsewhere that would permit works of such magnitude to be constructed for such low cost.

Conditions are certainly unusual which will permit the Government to have built free of cost such an essential part of the scheme as the diverting barrier at Daguerre Point, with a height of 30 ft., a width of 200 or 300 ft. and a length of 2½ miles. Of the south training wall, 2 miles will also be constructed in a similar manner. The dredges building these banks excavate to a depth of 60 ft.

Mr. Kieffer.

Even when gold values in the gravel of the river-bed fall below pay, as Marysville is approached, the construction of the south training wall should be effected at a very low figure by this method.

The great width of the Lower Yuba in its natural channel above Daguerre Point affords an unusual amount of storage for such a stream. Below Daguerre Point the width of 2 to 3 miles between levees would afford much greater room for subsidence outside of the training walls than is provided in the settling basin, although much of the most available part of this area on the south side will probably be mined by the dredges.

The elevation of the river-bed above the very flat valley lands makes the settling basin a very practical and easily accomplished part of the plan.

Barrier No. 1 is an interesting piece of engineering, the ultimate success of which seems to be assured. The length of the structure, its proposed height, the shifting character of the foundation, the floods to be passed, and the low cost for a virtually permanent structure under the conditions, are all circumstances that will cause its future to be watched with interest.

The question of maintenance, in a structure with such an exposure of concrete face to the wear of passing gravel, may prove serious. The author states that during the first winter the concrete facing to a depth of 2 in. was cut out in places.

The writer has had experience along this line upon a dam built across the American River. The dam above the rubble masonry was crested with T-rails on each edge and filled, for a depth of from 6 to 8 in., to the tops of the rails, with concrete composed of 1 part cement and 2 parts river gravel passing a 1½-in. screen. This was entirely cut out during the first season; was replaced and cut out the next year, and finally replaced with 2-in. pine plank set between the rails.

The greatest damage from such erosion occurs during the middle stages of the river, when only the finer materials are being transported. That this danger is anticipated in Barrier No. 1 is indicated by the construction of a wasteway, 400 ft. long and 4 ft. deep, sufficient to carry the ordinary river flow. This will undoubtedly mitigate the trouble, but will not entirely remove it.

There is no doubt that the ordinary aggregate of the concrete will withstand this wear better than a strong mortar.

That the plan in general outlined by the Commission for the control of the debris of the streams is probably the most feasible one, there can be little doubt. How far it can be made beneficial is a question, and is largely a matter of topography and money.

The settling basin, diverting barrier and Barrier No. 1 of the

Mr. Kieffer. Yuba project will probably impound from 40 000 000 to 45 000 000 cu. yd. of material. At the rate the lower Yuba basin filled, from 1899 to 1904, as given by the author, these works will be filled in about 10 or 15 years.

Other rivers, notably the American, afford poorer opportunity for impounding debris than the Yuba. In this case the river is frequently in flood when the Sacramento is not.

It is essential that all coarse material capable of being deposited in the slow-moving navigable streams should be held back if possible, but how far this can be done with the relatively small works capable of construction for the purpose only time will prove.

Mr. Harts. WILLIAM W. HARTS, M. AM. SOC. C. E. (by letter).—The application of the United States law concerning hydraulic mining, as was well pointed out by Mr. Riffle and Mr. Richards, affects only the water-sheds of the Sacramento and San Joaquin River systems, and in no other part of California has this kind of mining been interfered with under the law of March 1st, 1893. Much of the large quantity of gold-bearing gravel still in these areas is now almost completely sealed by the operation of this law, as the large mines are either unable to find settling and impounding basins of sufficient capacity, or are unable to pay the cost of their construction, which thus in most cases amounts to a prohibitive tax. There is promise, however, that some of the richer deposits may be dredged before long by machines similar to those now used on the Yuba and Feather Rivers, which have been developed to the point where their operation is practically as cheap as the cost of hydraulic mining per yard of material moved. When the profitable dredging ground in the beds of the rivers now being mined is nearing exhaustion, this industry will look about for new fields of operations, with the probable result that many old hydraulic banks now idle will be mined by the method of dredging. That this method has not already been tried, doubtless arises from several causes. Among these causes are the high first cost of a gold dredging machine compared with a hydraulic "giant" and plant (outside of the cost of the water), the lack of electric power at convenient places for operation, and the inaccessibility of many of the gravel deposits, to say nothing of the conservatism of the miners.

Furthermore, most of the easy mining ground of any great extent has been already attacked, and only the less rich or less accessible is left. Wherever these objections are not too serious, it is believed that in the future a movement may be expected looking toward the rejuvenation of mining in old hydraulic banks by this new process of dredging. No restriction would be placed on this by the Federal Government if the method of working in a closed pit should be followed, as is now being generally done, for no damage,

even from careless operation, could then result to the navigability of the lower rivers into which the mines drain. Mr. Harts.

As the long contest over hydraulic mining had unexpected benefits in directing new efforts toward quartz mining, which has been greatly developed in consequence, as was referred to by Mr. Richards, the need of finding new grounds for gold dredging operations will perhaps likewise cause a further exploitation of these old hydraulic pits and banks, with the hope of the production of much wealth therefrom.

As to the capacity of the present system of the Yuba River restraining works, mentioned by Mr. Galloway, this has been carefully worked out, but the destruction of all the records in the San Francisco fire makes it impossible to state these amounts definitely at present. The adopted plan of this work, however, contemplates other barriers above the one now under construction, should they be found necessary in the future, and furthermore, an enormous reserve basin is found between Daguerre Point and the present barrier, capable of storing many millions of yards. After the settling basin now being prepared on the south bank has been filled and its entrance permanently closed, an elevation of the sill at Daguerre Point will give a great volume of additional storage room between Daguerre Point and the barrier. In fact, this area is available nearly up to the level of the top of Daguerre Point. In addition, below Daguerre Point and lying between the training walls and the shores, further areas for storage of debris will be available. It has been estimated that the capacity far exceeds the quantity of debris in the upper cañons that may reasonably be expected to be brought down. The plan thus involves an elastic and progressive feature to make it adaptable to future needs.

Another extension of this plan is now receiving attention, and that is carrying the training walls down to Shanghai Bend in the Feather River. This, it is hoped, will add to the protection and safety of Marysville and afford a better outlet into the Feather River. With the detritus of the Yuba successfully restrained, the problem of the other streams is more easily met.

As to what is necessary for the other tributaries of the Feather, Sacramento and San Joaquin Rivers, the results of the survey of the Bear River has shown to the writer the desirability of adopting other and more direct methods of caring for the debris in the future work on these smaller rivers. The construction of two hydraulic dredges would cost not more than $300 000 for both, and the cost of the operation of both dredges would be about $5 000 per month each, or $60 000 per year, allowing a working season of 6 months. If the material excavated should then be placed on the levees along the Feather and Sacramento Rivers, it would serve many useful

Mr. Harts. purposes, and act both as a prevention and cure. This work, at the same time, would lower the river-bed, lower the high-water plane, diminish the danger and damage from floods, increase the navigability of the rivers at low stages, protect the adjoining land from overflow, and dispose of the debris already deposited, as well as prevent fresh sedimentation. In no other way can such a combination of benefits be secured at so low a cost. The application to the Bear River of the settling-basin method of retaining the debris would cost at least $300 000, and that for the American, perhaps $500 000. It is believed that for these rivers a far more efficient result would be reached by using the method of hydraulic dredging. With these amounts, at least eight years of work would be permitted, and a consequent removal of more than 6 000 000 cu. yd. of sand and silt to the side levees. Unquestionably, this would induce considerable natural scour at high water if the side levees were properly located. All work accomplished would thus tend to induce a natural improvement which would hasten the tendencies already noticed and referred to, and supplement the artificial work.

The dredge now at work for the United States in Oakland Harbor digs 70 000 cu. yd. per month, in much less favorable ground, so that the season's output of two dredges in mining silt ought to amount to nearly 1 000 000 cu. yd. It has only been within recent years that suction dredges have reached such a high efficiency, but it seems safe to say that for the less important rivers this method of caring for the debris will prove much more effective, now that the danger of the detritus of the Yuba River is in fair process of being corrected.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 1027.

DISPOSAL OF MUNICIPAL REFUSE, AND RUBBISH INCINERATION.*

By H. de B. Parsons, M. Am. Soc. C. E.

During the winter of 1902-03, the writer contributed to the informal discussion on "The Sanitary Disposal of Municipal Refuse."† At that time, some data were presented and a short description was given of the rubbish incinerator, built from the writer's plans, at the foot of West Forty-seventh Street, New York City. Subsequently, additional data were presented at the International Engineering Congress at St. Louis.‡

The writer now contributes to the Society further data on the subject of municipal refuse, which he hopes will prove both interesting and instructive, and a description of the rubbish incinerating plant built on Delancey Slip, Borough of Manhattan, City of New York, by order of John McGaw Woodbury, Assoc. Am. Soc. C. E., Commissioner of the Department of Street Cleaning, together with a description of the adjoining electric lighting station (built by order of the Department of Bridges), which utilizes the heat produced from the incineration of the rubbish to light the Williamsburg Bridge.

* Presented at the meeting of June 6th, 1906.
† *Transactions*, Am. Soc. C. E., Vol. L, p. 95.
‡ *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, p. 263.

This combined plant was built by the Departments of Street Cleaning and of Bridges for the benefit of the municipality, and each department retained the writer as Consulting Engineer to prepare the plans and superintend the construction. The Department of Street Cleaning gains by having the rubbish collections reduced in bulk and transformed into ash which is valuable for land-fills, and by the improved sanitary conditions caused by completely burning the beds, bedding, and old furniture, which are often germ laden, and thus preventing them from ever returning to the city. The Department of Bridges saves in the cost of lighting the bridge by using the heat of combustion for the generation of electricity. To show how well these objects have been realized is part of the subject of this paper.

General Data.

Portions of the data here given have been taken from the papers mentioned previously. The repetition has been made in order that the new data presented, together with the former data, may be brought into this contribution, and the whole rendered as complete as possible.

A general division and subdivision of the materials is given in Table 1. The scheme classifies the wastes of a municipality and places them in three divisions, the basis of classification being chiefly dependent upon the method of handling, as practiced by "usage and custom." The first and second divisions—the "fluid and semi-fluid refuse" and the "general refuse"—consist of that portion for which the authorities should make provision, as the people cannot be made, or entrusted, to dispose of them in a sanitary manner. The third division includes those wastes which are cared for by private or special service—either by private contract or by the parties creating the refuse. The item of dead animals, however, is a possible exception, and, in large cities, the authorities should make some provision for their removal.

As uniformity in practice is lacking, the scheme for disposal adopted in any city will not always agree with the method of division for classification as given in Table 1. Some cities provide for the disposal of only a part of the general refuse, while others care for the whole.

The general refuse, which is the subject of this paper, and for

FIG. 1.—REFUSE DUMPED DIRECTLY INTO SCOWS.

FIG. 2.—FRONT OF INCINERATOR BUILDING AT DELANCEY SLIP.

FIG. 3.—MATERIAL PICKED OUT, BALED AND READY FOR SHIPMENT.

which the community should provide a method for collection and disposal, is separated into five classes:

I.—Ashes;
II.—Garbage;
III.—Rubbish;
IV.—Street-sweepings;
V.—Snow.

TABLE 1.—Classification of City Wastes.

City Waste Materials	Fluid and Semi-fluid Refuse	Sewage	House Sewage.
			Street, Roof and Area Drainage.
			Night Soil.
	General Refuse	Ashes	Steam Ashes.
			Household Ashes.
		Garbage	Animal Matter.
			Vegetable Matter.
			Meat and Bones.
			Fruit.
		Rubbish	Paper.
			Wood.
			Rags and Bedding.
			Leather and Rubber.
			Metals.
			Bottles, Glass and Crockery.
			Sweepings from Buildings.
		Street-Sweepings	Animal Manure.
			Pavement Dirt.
			Droppings from Carts.
			Materials from Building Construction.
			Some Rubbish and Leaves.
		Snow	
	Trade Refuse		Cellar Excavations.
			Materials from Building Construction.
			Stable Manure.
			Market Offal.
			Slaughter-house Offal.
			Dead Animals.

The characteristics of the general refuse divisions can be stated as follows:

I.—Ashes.—Ashes consist of silica, oxide of iron, potash, alumina, lime, magnesia, soda, barium, phosphorus in phosphates, sulphur in sulphates, etc., and of unburned coal.

Ashes weigh about 1 350 lb. per cu. yd., varying from 1 200 to 1 500 lb., and can be divided into two grades or classes, *viz.*: "steam-ashes" and "household-ashes."

The steam-ash is that which is obtained from coal burned under steam boilers, in those industrial works where care is taken to have

the combustion reasonably perfect. The amount of combustible in steam-ash varies from about 11 to 50%,* increasing as the size of the coal decreases and as the coal is of a less caking character. This amount of combustible in the steam-ash represents a loss of unburned coal in the ash of from 2 to 33% of the fuel. A fair general average could be assumed at 26% by weight of the ash, or at 4% of the total coal used. With mechanical stoking grates, for the small sizes of anthracites and for the anthracites of a friable nature, the amount of combustible in the ash would probably exceed the average just stated. For the ordinary anthracites, the average would approximate 30%; and for the bituminous coals, the average would be about 24 per cent. For the caking coals, the average would be excessive. The loss would also be greater than the foregoing average for coals containing large percentages of earthy matter.

Table 2 shows the percentage of combustible matter found in the ash, as reported in a number of steam boiler trials, being the percentage by difference between the ash as determined by analysis and that as recovered from the ash-pit and weighed.

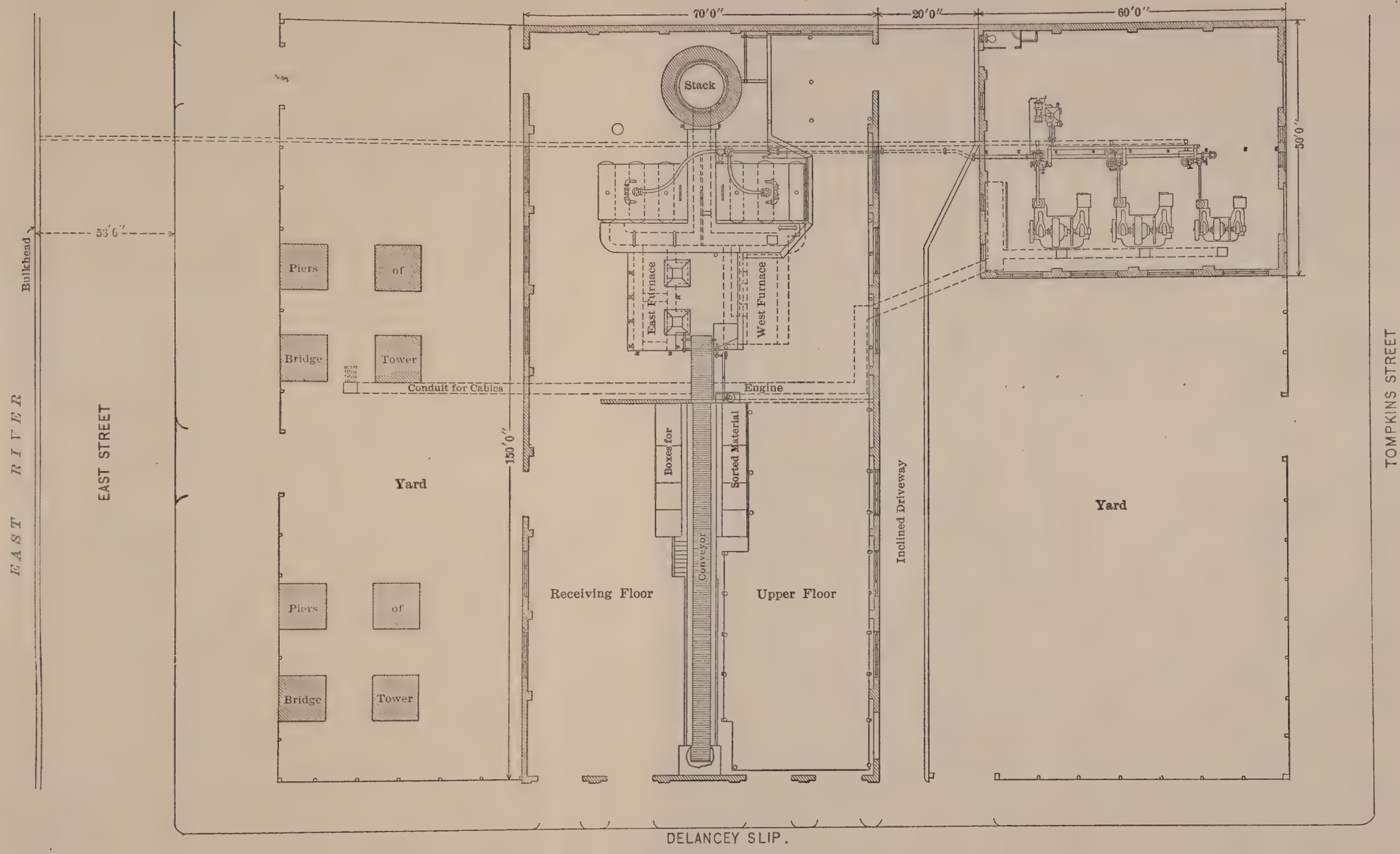
The household-ash is that which is obtained from coal burned in house-heaters, ranges, stoves and open fire-places. The coal used is almost entirely of those sizes which will pass through a 4½-in. screen and over a 1¼-in. screen, with the exception of the cannel coal consumed in open fire-places. While this cannel naturally burns to a clean ash, still a large quantity falls through the basket grates; and, furthermore, as the coal is of such a soft nature, much of it, in handling, crumbles into dust and small pieces, and is thus rendered too small for household use. All this combustible portion finds its way into the ash collections. On the continent of Europe, a smaller proportion of unburned coal is found in the household-ash collections than in the United States, as the use of closed stoves for household purposes is so common. In England, the proportion is greater, on account of the numerous open fire-places.

For the past twelve years, the writer has kept a complete coal record for his private dwelling. The cannel coal dust which collected between August, 1896, and August, 1902, and which had to be discarded from the household, amounted to 8.3% of the weight of cannel coal purchased. Assuming that this cannel coal dust con-

*See "Steam Boilers," by H. de B. Parsons, p. 23.

PLATE XII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVII, No. 1027.
PARSONS ON
REFUSE DISPOSAL.

RUBBISH INCINERATOR AND ELECTRIC LIGHTING STATION, GENERAL ARRANGEMENT OF PLANT.



tained 4% of true ash, then the combustible portion discarded was about 8% in addition to that which was contained in the ashes from the grates.

TABLE 2.—Unburned Coal in Steam-Ash.

Kind of coal.	Percentage of Ash.		Percentage of Combustible.	
	By analysis.	By boiler trial.	Based on ash.	Based on coal.
	Per cent.	Per cent.	Per cent.	Per cent.
* Cumberland Bituminous	5.0	7.5	33.3	2.5
* Pocahontas "	4.0	10.0	60.0	6.0
* Cumberland "	6.1	8.1	24.7	2.0
* Clearfield "	7.6	10.1	24.7	2.5
* Cape Breton "	7.8	9.4	17.0	1.6
† George's Creek "	9.0	10.6	15.1	1.6
† W. V. Pocahontas "	11.1	11.9	6.7	0.8
† George's Creek "	9.3	10.5	11.4	1.2
† Buckwheat, No. 1, Anthracite	17.5	22.5	22.2	5.0
† Buckwheat " "	16.2	24.9	35.0	8.7
* Chestnut, No. 2, " "	12.0	16.0	25.0	4.0
* Chestnut " "	11.8	19.3	38.8	7.5

General average..... 26 per cent.
Average, Anthracite..... 30 "
" Bituminous..... 24 "

* Barrus on "Boiler Tests." page 251.
† From tests made by D. S. Jacobus.

The proportion of cannel to anthracite used was 1 to 7.6 or 13 per cent. By analysis, the combustible matter in the cannel coal ashes was 21.8%, making a total combustible of about 29.8 per cent. Also, by analysis, the combustible matter in the anthracite ash was 13.4 per cent. Therefore, the real average combustible in the ash discarded from the house, which can be taken as an average of similar houses in the residential portion of New York City, amounted to 15.3 per cent.

The writer had careful analyses made in February and March, 1904, of some household ashes and of samples of ash obtained directly from some of the dumps of the City of New York, with the results shown in Table 3. Therefore, it would seem safe to state that in a city like New York, the mixed ash collections from all quarters contain from 30 to 35% of combustible matter.

All the analyses of ash were made by Messrs. Simonds and Wain-

wright, Analytical Chemists. The samples from the city dumps were made by taking a shovelful from alternate carts as they drove on the dumps during the major part of one day at a dump. The pile was mixed and quartered, and the final sample, of from 12 to 15 lb., was crushed in a mill and again quartered.

TABLE 3.—Analyses of Ashes.*

	Moisture at 102° C.	Pure ash.	Combustible matter.	Total.
Household Ash.	Per cent.	Per cent.	Per cent.	Per cent.
Coal burned in a stove, anthracite, size "stove No. 2"	0.36	90 81	8.83	100.00
Coal burned in a hot-air furnace, anthracite, size "egg"	0.06	86.50	13.44	100.00
Coal burned in an open grate, English cannel, size "6-in. cubes"	0.64	77.53	21.83	100.00
Mixed Ashes.				
Samples from New York City Dumps...	1.69	62.19	36.12	100.00
Clinton Street	0.80	67.43	31.77	100.00
Stanton Street	0.83	63.73	35.44	100.00
West Forty-seventh Street	1.11	64.45	34.44	100.00
Average				

* Made for the writer by Messrs. Simonds and Wainwright.

II.—Garbage.—Garbage is, by far, the most important division of general refuse, because it is the most difficult to handle without causing annoyance, is unsightly, and is likely to become putrescible and diffuse offensive odors. On the other hand, it has a distinct commercial value, due to the value of the by-products which may be obtained from it. Whether this value is worth saving, in practice, is still a moot question, and no unity of sentiment exists, although much has been written concerning its treatment.

Its composition differs according to the season of the year, as well as the location of the city and the character of the district from which it is collected. Garbage contains a large proportion of water, the amount varying from about 50% to more than 80 per cent. During the fruit and green vegetable seasons the amount of water is always large, while in the ordinary dwelling-house garbage in winter the amount is nearer the lower limit.

In Table 4 are given some analyses of the composition of garbage. Analysis *A* was made by the Sanitary Bureau of the Board of Health of the City of New York, in October, 1897. The sample

FIG. 1.—BELT-CONVEYOR AND SORTING-BOXES.

FIG. 2.—END OF BELT-CONVEYOR AT PLATFORM OVER FURNACES.

was taken from the garbage as received on the barges or scows for final removal, and may be considered an average of the collections from many and various sources. It was practically pure garbage.

TABLE 4.—Analyses of Garbage.

	New York City. *A.*	Brooklyn, N. Y. *B.*	Trenton, N. J. *C.*	United States. *D.*	England. *E.*	Berlin. *F.*
	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.	Per cent.
Water	65.90	71.00	80.00	70.00	65.00	60.00
Animal and vegetable solids	25.62	20.00	16.80	20.00	24.00	30.00
Grease	.....	2.00	.....	3.00	2.00	2.00
Paper	.....	7.00	2.40	7.00	9.00	8.00
Rags, etc	.....		0.60			
Boxes	.....		0.20			
Mineral matter	8.48	.....	.....	.....	.....	.....
Totals	100.00	100.00	100.00	100.00	100.00	100.00
Fat	7.07	.....	.....	.....	.....	.....
Total nitrogen	0.86	.....	.....	.....	.....	.....
Phosphoric acid, P_2O_5	0.07	.....	.....	.....	.....	.....
Potash, K_2O	0.30	.....	.....	.....	.....	.....

A. Dept. of Health, October 15th, 1897.

B. Report, Brooklyn Board of Health, 1896.

C. Transactions, Am. Soc. C. E., Vol. L, p. 128.

D, *E* and *F.* International Engineering Congress, St. Louis, *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, p. 263.

Analysis *B* is an analysis of Brooklyn summer garbage, as stated in the report of Joseph B. Taylor.*

Analysis *C* was made by Theodore Horton, Assoc. M. Am. Soc. C. E., of garbage as collected in Trenton, N. J., during the summer of 1902. It was obtained by sorting ½-ton samples and weighing and averaging.

Analysis *D* gives the composition of average American garbage, as stated by MacDonough Craven.

Analyses *E* and *F* are quoted on the authority of Rudolph Hering, who obtained his figures from W. F. Goodrich, of London, and Messrs. Bohm and Grohn, of Berlin.

All the figures in the table are given in percentages, and do not indicate more than the relative proportions of the elements reported. The actual quantities collected in certain cities are stated later.

* Board of Health Report, Brooklyn, 1896.

Garbage, as ordinarily collected, contains some rubbish, such as paper, cans, boxes, etc. Even in those cities where a primary separation system is in vogue, this admixture of rubbish may amount to 5 or 6% or more by weight.

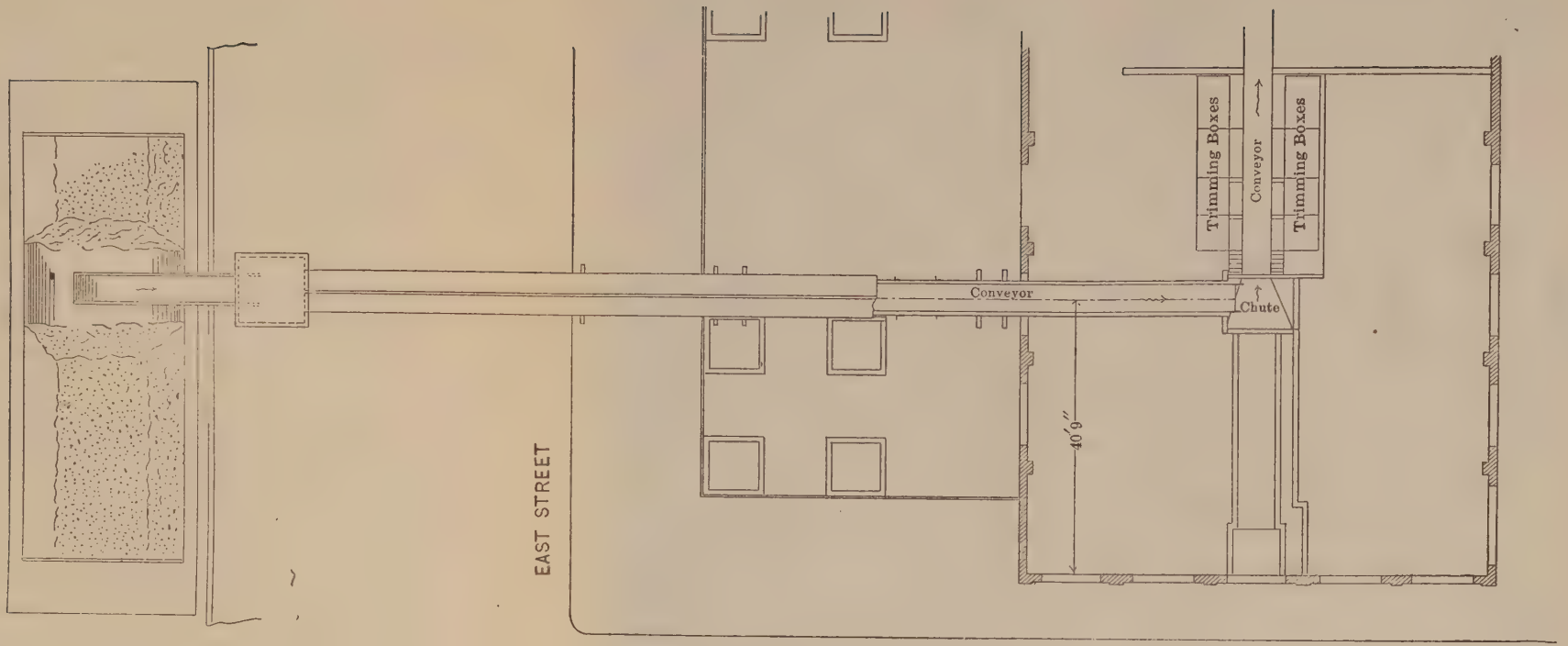
Pure garbage weighs about 1 100 or 1 200 lb. per cu. yd., when loosely collected.

III.—Rubbish.—Rubbish is discarded trash, principally, of all kinds of paper, wood, rags, mattresses, bedding, boxes, chairs, sofas, barrels, leather, old shoes, rubber, tin cans, metal scraps, bottles, broken glass, crockery and the like. It is a most heterogeneous aggregation, and contains all the household wastes that cannot be classified as ashes or garbage. Every conceivable kind of rubbish waste is discarded from the houses or found in the street-sweepings; but the distinction between these classes of refuse is that such of the material as is collected from the houses strictly belongs to the rubbish classification, and such as is obtained from the streets to the street-sweeping classification.

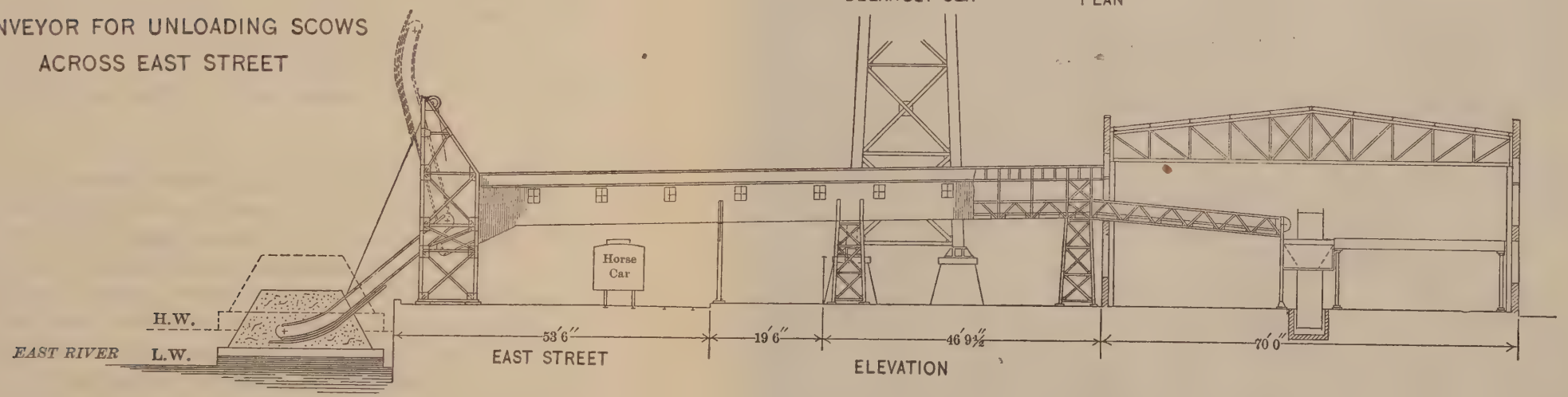
Rubbish, as ordinarily piled in the carts, or without extra packing, weighs from 130 to 225 lb. per cu. yd. In Boston, Mass., the average weight, as delivered at the Atlantic Avenue collecting and incinerating station, was 202 lb. per cu. yd. In New York the average weight at the Thirtieth Street dump was 143 lb. per cu. yd.; at the Forty-seventh Street dump, 141.1 lb., and at the Delancey Slip incinerator, 139 lb.

As rubbish contains the dirt and dust from sweepings, cast-away bedding and rags, it is likely to harbor germs of disease, and should be taken to the place for final disposition as directly as possible, so that dry, germ-bearing dust may not be scattered among the people of the community. In other respects, the rubbish collections may be termed clean, and are not especially disagreeable to handle.

Some analyses of rubbish are given in Table 5. The figures in the first column are measurements made by the writer in 1905, and in the second and third columns by F. L. Stearns, Assoc. M. Am. Soc. C. E., of the Department of Street Cleaning of New York City, in 1904. These figures, together with those for Boston, are the percentages by weight of the marketable portion only, that is, that portion which was picked out and sold. The remainder was valueless, and was not classified, but was composed of the same ele-



CONVEYOR FOR UNLOADING SCOWS
ACROSS EAST STREET



ments as mentioned, together with bedding, mattresses, furniture, etc.

The smell arising from a collection of rubbish is not offensive, and the mass does not decompose. At times the collections contain some garbage, which on decomposing gives the mass a decided smell.

Rubbish, like garbage, has an inherent value. A considerable portion of the mass can be sorted out and sold at a profit. In consequence, the collections are often picked over, for which privilege contractors can be found.

The privilege of picking the rubbish at the dumps in the Boroughs of Manhattan and the Bronx brought $71 000 for 1903. For a week in 1904 the amounts varied from $1 175 to $1 920. The collections in 1903 were 126 188 tons.

At the Atlantic Avenue incinerating plant in Boston, the amount picked out and sold averages about 25½% by weight. In New York, at the Thirtieth Street dump, the amount is about 43%; at the Forty-seventh Street dump, about 48%; and at the Delancey Slip station, about 32 per cent. The two latter places are equipped with conveyors which carry the material between the rows of pickers. At the other city dumps the amount picked out is not as large. As the Delancey Street station is equipped with an incinerating plant using the heat for steam generation, only about 30% is picked out, as the remainder is required for fuel, although with the conveyor and sorting opportunity it would be easy to pick out 50 per cent. The amount picked out varies with the market price for the paper, metals, etc.

IV.—Street-Sweepings.—The sweepings contain two constituents of value, namely, manure and paper. When these are mixed with the street dirt, they are valueless. If the manure could be collected separately, it might be sold for fertilizer, or be sold for the same purpose to the collectors of stable manures.

Considerable rubbish, often of the dirtiest sort, is found in public thoroughfares, where it has been thrown by a careless and shiftless population. These materials—rubbish, manure, paper, and dirt, the latter coming from the pavements and, therefore, varying with the kind in use—form the major part of the street-sweepings proper. There are some house sweepings collected in the streets, but this addition aggregates but a small part. In cities in which the streets

are lined with trees, a considerable quantity of leaves is collected in the proper season, which increases the bulk materially.

TABLE 5.—Composition of Rubbish.

Percentages by Weight.

Component Parts.	Percentage Picked Out as Marketable.				Percentage of Total Composition.		
	City of New York.			Boston.	New York.	London.	Berlin.
	Delancey Slip station. (Parsons.)	Thirtieth St. dump. (Stearns.)	Forty-seventh St. dump. (Stearns.)	Atlantic Ave. station. (Morse.)	(Craven)*	(Russell)*	(Bohm and Grohn.) *
	%	%	%	%	%	%	%
Stoneware	.....	.....	.....	.....	.....	5.0	33.5
Rags	4.6	.....	2.78	0.76	15.5	3.6	6.3
Rubber	.....	.....	.....	.....	0.1	.....	.....
Leather	.....	.....	.....	.....	1.8	.....	3.8
Straw	.....	.....	.....	.....	.....	29.7	19.7
Wood	.....	7.3	8.91	.....	1.4	.....	2.2
Metals	0.86	1.3	4.10	0.12	3.3	9.2	4.2
Glass	.....	1.4	0.76	0.35	2.9	13.1	7.0
Bagging	.....	.....	0.39	.....	.....	.....	.....
Carpets	.....	.....	0.57	.....	.....	.....	.....
Shoes	.....	.....	0.39	.....	.....	.....	.....
Hats	.....	.....	0.03	.....	.....	.....	.....
Rope and String	.....	.....	0.23	0.12	.....	.....	.....
Paper	25.4	33.3	.....	23.90	75.0	39.4	23.3
Newspaper	.....	.....	10.94	.....	.....	.....	.....
Manila	.....	.....	2.64	.....	.....	.....	.....
Pasteboard	.....	.....	10.35	.....	.....	.....	.....
Mixed	.....	.....	6.16	.....	.....	.....	.....
Books	.....	.....	0.55	0.24	.....	.....	.....
Total marketable	30.86	43.3	48.80	25.49	.....	.....	.....
" worthless	69.14	56.7	51.20	74.51	.....	.....	.....
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0

*International Engineering Congress, *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, Paper by Rudolph Hering, M. Am. Soc. C. E.

The sweepings are combustible. The weight varies from 800 to 1 400 lb. per cu. yd., as much depends on the dryness of the weather at the time of collection.

Table 6 gives some idea of the composition of the street-sweepings.

The writer entered into correspondence with a number of cities in order to obtain data in regard to the annual collection of general refuse. It was found that many of the cities did not make separate collections of the different classes of refuse, and that many which did make separate collections did not keep records, or if they did

keep records, did not keep them with sufficient accuracy to be of service for comparison. For instance, Louisville, Ky., published in 1903 a very elaborate report in which the collections were given in cart loads. As no reference was made to the size of the carts, or their average loading, it was not possible to classify these collections.

TABLE 6.—Composition of Street-Sweepings.*
Percentages by Weight.

Component parts.	New York. (Craven.)	Washington. (Wiley.)	Berlin. (Vogel.)	London. (Letheby)†
Moisture	37	35	39	35
Organic matter	31	20	23	36
Ash	32	45	38	29‡
	100	100	100	100
Proportion of organic matter to ash	1 : 1	1 : 2.2	1 : 1.6	1 : 0.8

* From U. S. Dept. of Agriculture "Fertilizing Value of Street Sweepings," Bulletin No. 55, Division of Chemistry. International Engineering Congress, *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, Paper by Rudolph Hering, M. Am. Soc. C. E.
† In dry weather.
‡ Powdered stone and abraded iron.

Some of the cities kept a record in yardage, and, in order to transform yardage into weight, the following schedule was used:

Weight of garbage	1 150	lb. per cu. yd.
" " street-sweepings	850	" " " "
" " ashes	1 350	" " " "
" " rubbish	200	" " " "

Table 7 gives statistics of the annual collection of general refuse, in tons of 2 000 lb.

Table 8 gives the monthly collections, in pounds per capita per day, in those cities where the records were available. This table shows some variation in regard to the amounts, which is especially noticeable in the column, "Average per day." This variation is caused, to some extent, by irregularity in classifying the collections —for instance, in some cities, rubbish and ashes are collected with the garbage—thus, in Philadelphia the record shows that the quantity of garbage per capita per day is nearly 1.2 lb., while in the other cities it varies from about 0.3 to 0.8 lb., while the ashes and rubbish collections are low when compared with the others.

TABLE 7.—Annual Collection of City Refuse, in Tons of 2 000 lb.

City.	Year.	Population.	Garbage.	Street-sweepings.	Ashes.	Rubbish.	Totals.
Borough of Manhattan	.....	1 917 676	184 275	*B*	1 405 606	120 434	1 710 315
" " Bronx	.....	268 341	13 475	.....	109 699½	5 754	128 928
" " Brooklyn	.....	1 291 597	75 675	.....	501 888½	39 246	616 809
Total of three Boroughs of New York	1903.	*A* 3 477 614	273 425	.....	*D* 2 017 193⅔	*C* 165 434	2 456 053
Buffalo, N. Y.	June 30, 1903.	*E* 391 148	*F* 23 803	*B*	*F* 150 469	*F* 15 750	190 022
Philadelphia, Pa. *H*	Dec. 31, 1903	*E* 1 385 549	301 643	93 044	425 650	13 975	834 312
Milwaukee, Wis. *H*	.....	*E* 307 854	*K* 30 441	.....	.....	.....	.....
Cincinnati, Ohio *H*	1903.	381 200	21 600	39 362	142 507	.....	.....
Washington, D. C.	Dec. 31, 1903.	*E* 296 182	*M* 33 664	*L* 31 810	*L* 83 078	*L* 14 150	.....
Newark, N. J. *H*	.....	*E* 272 149	15 152	33 496	*G* 208 050	*G*	.....
Cleveland, Ohio *J*	Dec. 1, 1904.	425 000	43 680	35 036	.....	.....	.....
Pittsburg, Pa.	.....	320 000	*O* 47 000	.....	.....	.....	.....
St. Louis, Mo.	Dec. 31, 1903.	*E* 622 350	62 400	.....	*N*	*N*	.....

A. Board of Health census, 1903.
B. Street-sweepings included with "ashes."
C. The Department of Street Cleaning estimates the weight of a cubic yard of rubbish and paper at 237 lb. F. L. Stearns, Assoc. M. Am. Soc. C. E., measured and weighed 96 loads in which 1 cu. yd. weighed 140 lb. This figure has been used in reducing the yardage to pounds, one load = 1 000 lb.
D. Including "steam" ashes from manufacturing plants.
E. United States census.
F. Report, Department of Public Works, Buffalo, 1903.
G. Rubbish and paper included with ashes.
H. Bureau of Street Cleaning, or Board of Public Works, 1903.
J. Bureau of Street Cleaning, or Board of Public Works, 1904.
K. Average for 1901 and 1903.
L. Average for 1903–1904.
M. Garbage for 1903; street-sweepings, ashes and rubbish averaged for 1903–1904.
N. Collections not made by the city.
O. Contractor's statement for 1903.

TABLE 8.—Collection of City Refuse, in Pounds per Capita per Day.

City.		Classification.	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Average per Day.
New York City.	Bor. Manhattan. A	Garbage	0.354	0.372	0.477	0.540	0.567	0.613	0.596	0.622	0.628	0.613	0.505	0.416	0.525
		Street-Sweepings													
		Ashes	5.282	5.089	4.778	4.123	3.588	3.528	3.326	2.975	3.167	3.437	4.017	5.082	4.016
		Rubbish	0.294	0.277	0.311	0.345	0.389	0.391	0.385	0.351	0.365	0.377	0.337	0.317	0.345
		Totals													4.886
	Bor. Bronx. A	Garbage	0.185	0.194	0.249	0.282	0.296	0.319	0.311	0.324	0.328	0.320	0.263	0.217	0.274
		Street Sweepings													
		Ashes	2.932	2.893	2.653	2.294	1.994	1.962	1.843	1.651	1.762	1.912	2.295	2.822	2.241
		Rubbish	0.100	0.094	0.106	0.118	0.133	0.133	0.131	0.120	0.124	0.128	0.115	0.108	0.1175
		Totals													2.6325
	Bor. Brooklyn. A	Garbage	0.216	0.236	0.280	0.330	0.344	0.374	0.364	0.380	0.383	0.374	0.308	0.265	0.321
		Street Sweepings													
		Ashes	2.789	2.684	2.520	2.176	1.892	1.863	1.754	1.565	1.673	1.813	2.121	2.680	2.127
		Rubbish	0.141	0.134	0.150	0.167	0.188	0.189	0.186	0.170	0.176	0.182	0.162	0.153	0.1665
		Totals													2.6145
Buffalo, N. Y.	B	Garbage	0.244	0.230	0.268	0.279	0.299	0.333	0.389	0.405	0.486	0.445	0.340	0.290	0.334
		Street-Sweepings													
		Ashes	2.762	2.662	2.501	2.161	1.874	1.850	1.741	1.552	1.660	1.800	2.101	2.660	2.110
		Rubbish	0.188	0.178	0.200	0.222	0.250	0.252	0.247	0.226	0.227	0.242	0.217	0.203	0.221
		Totals													2.665
Philadelphia, Pa.	A	Garbage	0.878	0.849	0.753	1.011	0.812	0.976	1.327	1.340	2.494	1.479	1.167	1.216	1.192
		Street-Sweepings	0.281	0.249	0.350	0.437	0.404	0.355	0.463	0.349	0.484	0.373	0.370	0.310	0.368
		Ashes	2.075	2.106	1.817	2.136	1.513	1.351	1.465	1.101	1.459	1.295	1.608	2.278	1.683
		Rubbish	0.071	0.061	0.047	0.061	0.052	0.050	0.058	0.046	0.062	0.048	0.049	0.055	0.055
		Totals													3.298
Milwaukee, Wis.	B	Garbage	0.539	0.514	0.511	0.491	0.514	0.594	0.636	0.625	0.588	0.576	0.489	0.422	0.542

City.	Classification.	Jan.	Feb.	March.	April.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Average per Day.
Cincinnati, Ohio. *A*	Garbage													0.310
	Street-Sweepings													0.566
	Ashes													2.048
	Rubbish													
	Totals													2.924
Washington, D. C. *A*	Garbage	0.511	0.512	0.586	0.641	0.600	0.577	0.666	0.871	0.819	0.733	0.552	0.543	0.635
	Street-Sweepings													0.597
	Ashes													1.557
	Rubbish													0.264
	Totals													3.053
Newark, N. J. *B*	Garbage													0.305
	Street-Sweepings													0.674
	Ashes													4.189
	Rubbish													
	Totals													5.168
Cleveland, Ohio. *B*	Garbage													0.563
	Street-Sweepings													0.451
	Ashes													
	Rubbish													
	Totals													
Pittsburg, Pa.	Garbage													0.805
	Street-Sweepings													
	Ashes													
	Rubbish													
	Totals													
St. Louis, Mo.	Garbage													0.549
	Street-Sweepings													
	Ashes													
	Rubbish													
	Totals													

A—Report, Department of Street Cleaning. *B*—Report, Department of Public Works.

TABLE 9.—AVERAGE COLLECTIONS PER CAPITA PER DAY.

	Pounds.	Yards.	Percentage.	
			By weight.	By volume.
Garbage	0.5296	0.000460	14.12	11.77
Street-Sweepings	0.5310	0.000625	14.15	15.99
Ashes	2.4960	0.001849	66.54	47.31
Rubbish	0.1948	0.000974	5.19	24.93
Total	3.7514	0.003908	100.00	100.00

Table 9 was made by taking the averages of the figures in Table 8. Some of the cities collected two of the divisions together and gave the result under one division; thus, New York collected the street-sweepings and ashes together and gave the total as ashes.

Table 10 was made up by using the foregoing percentages to divide those double collections, and gives the averages for all the cities. It is, therefore, more nearly correct than Table 9.

TABLE 10.

	Pounds.	Yards.	Percentage.	
			By weight.	By volume.
Garbage	0.5296	0.000460	15.27	12.13
Street-Sweepings	0.4992	0.000587	14.39	15.48
Ashes	2.2290	0.001696	64.28	44.71
Rubbish	0.2101	0.001050	6.06	27.68
Total	3.4679	0.003793	100.00	100.00

The average collections per capita per day for the cities mentioned are given in Table 9.

The figures given in Table 8, for the monthly collections of general refuse, in pounds per capita per day, are shown graphically in Fig. 1.

In the same manner, Figs. 2 to 4 show graphically the collections of ashes, garbage and rubbish. All these diagrams show a characteristic maximum about January and a minimum about

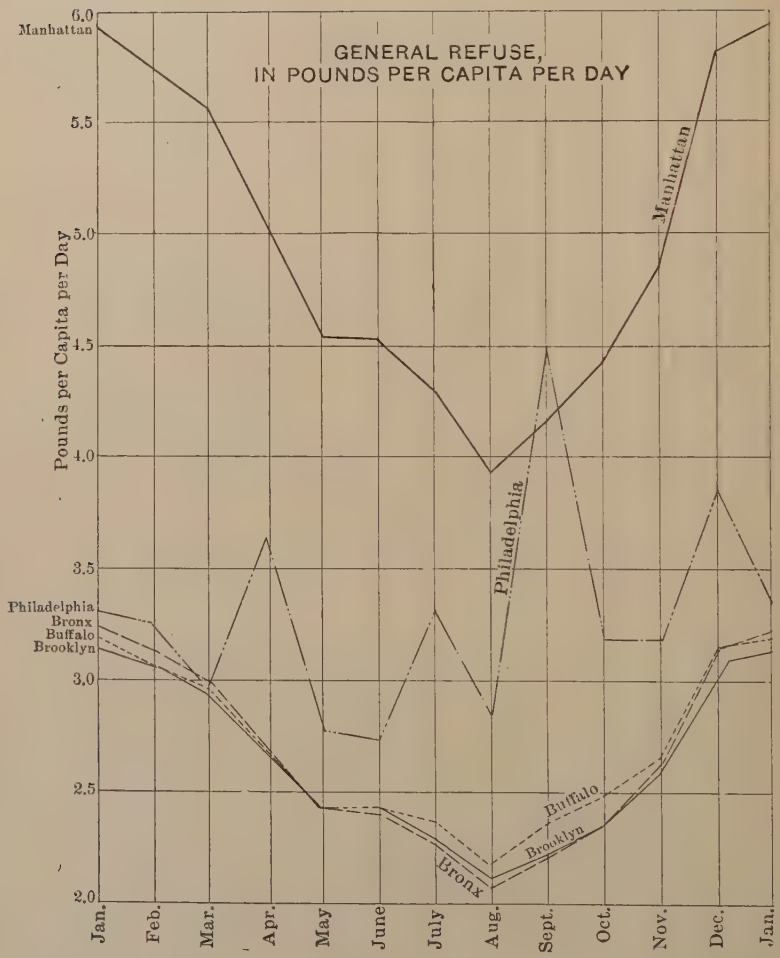

GENERAL REFUSE,
IN POUNDS PER CAPITA PER DAY
Pounds per Capita per Day
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
Manhattan
Philadelphia
Bronx
Buffalo
Brooklyn
Manhattan
Philadelphia
Buffalo
Brooklyn
Bronx
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.

FIG. 1.

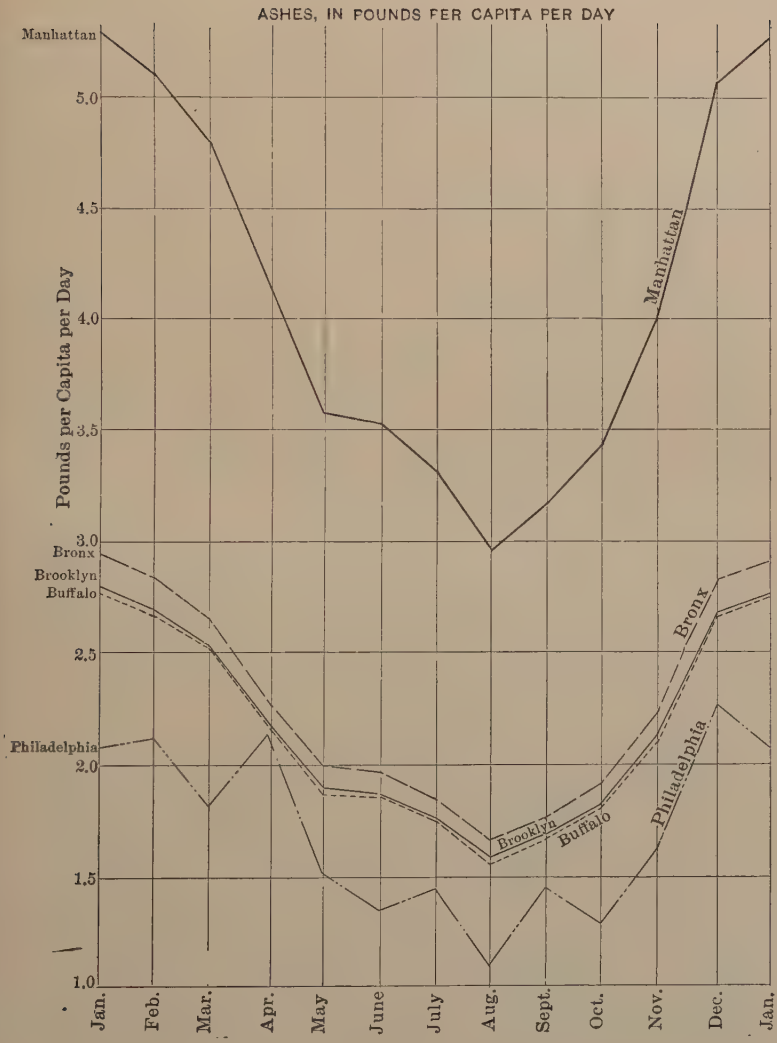

ASHES, IN POUNDS PER CAPITA PER DAY
Pounds per Capita per Day
Manhattan
5.0
4.5
4.0
3.5
3.0
Bronx
Brooklyn
Buffalo
2.5
Philadelphia
2.0
1.5
1.0
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.
Manhattan
Bronx
Brooklyn
Buffalo
Philadelphia

FIG. 2.

August. The maxima and minima are evidently caused by the collection of ashes, because the rubbish collections, while much more uniform than garbage or ashes, are in maximum during the summer months, and the garbage is in maximum about September.

Fig. 5 shows the monthly collections, in pounds per capita per day, for the Borough of Manhattan, New York City.

In studying these diagrams, it must be borne in mind that the ash collections as reported for the Borough of Manhattan, New York City, also contain the street-sweepings. The curve, therefore, representing Manhattan in Fig. 2 is too high by just the amount of the street-sweepings, which are not reported separately. In the same manner, in Fig. 3, the curve representing garbage collections for Philadelphia is too high, as some rubbish and ashes are collected and reported under the head of garbage.

In Fig. 1, the curve representing Manhattan (as well as the figures in Table 8 for the Borough of Manhattan) is higher than those of the other cities reported, which can be accounted for by the fact that the territory of the Borough of Manhattan is completely built over, and the whole population is served by the collection carts. In the other cities there are outlying districts which are included in the population, but which are not served by the collection carts. It is probable, therefore, that the actual figures for Manhattan are more accurate than for some of the other places.

The Fuel Value of Refuse.

The value of refuse as a fuel depends on the combustible matter which it contains. There does not appear to be much difference in the general make-up of the combined refuse as collected in America and Europe. It differs more in quantity than in complection.

Garbage will burn, as can be proved by throwing it into a range or stove. It requires enough heat to evaporate the water, and then the dry matter will burn of itself and assist in drying the next charge. This dried portion will generate about 7 500 B. t. u. per lb.

The combustible in the ashes has a total heat of combustion varying from, say, 10 000 to 14 000 B. t. u. per lb. Therefore, the dry ash, on a basis of 25% of combustible, will generate about 3 000 B. t. u. per lb. As the ashes, according to the analyses given

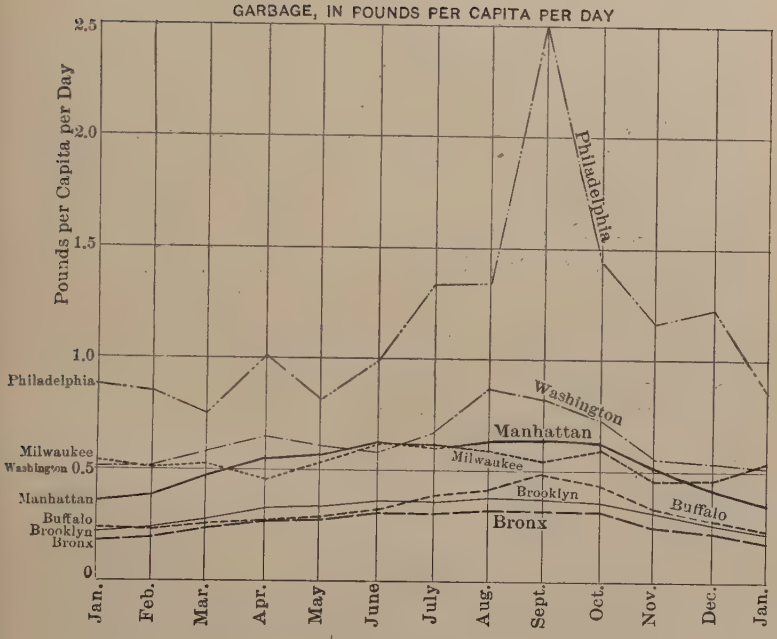

GARBAGE, IN POUNDS PER CAPITA PER DAY
Pounds per Capita per Day
2.5
2.0
1.5
1.0
0.5
0
Philadelphia
Washington
Manhattan
Milwaukee
Brooklyn
Buffalo
Bronx
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.

FIG. 3.

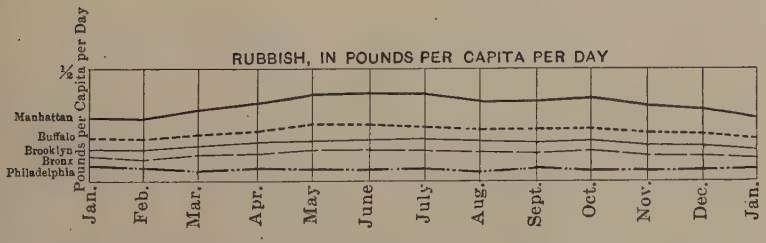

RUBBISH, IN POUNDS PER CAPITA PER DAY
Pounds per Capita per Day
½
Manhattan
Buffalo
Brooklyn
Bronx
Philadelphia
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.

FIG. 4.

before, contain an average of more than 25% of combustible, there is an allowance at this latter figure for some moisture.

The rubbish, being composed principally of paper, rags and wood, has a total heat of combustion varying from, say, 5 000 to 7 500 B. t. u. per lb.

The street-sweepings contain some combustible material, which can be taken as having a total heat of combustion of about 6 000 B. t. u. per lb. The remainder of the street-sweepings consists of incombustible material and water. If it be assumed that 33% is incombustible, 42% combustible, and 25% water, the burning of the street-sweepings would generate about 2 000 B. t. u. per lb., a probably safe figure.

Assume that the combined collection is composed as shown in Table 11:

TABLE 11.

Classification.	City collections.	Used as fuel.
Garbage	15.0% (70% moisture)	22.9%
Ashes	64.0% (25% combustible)	48.7%
Rubbish	7.0% (33⅓% sorted out)	7.1%
Street-sweepings	14.0%	21.3%
Totals	100.0%	100.0%

Also, assume that 33⅓% of the rubbish is sorted out and only half the ashes are delivered to the furnace, so that the percentage of the constituents as thrown into the furnace would be in the ratio stated in the last column of Table 11. From these assumptions, a heat balance can be worked out, as shown in Table 12.

The combustible portion is: garbage, 0.0687; ashes, 0.0913; rubbish, 0.064 (allowing 10% for the incombustible portion); and street-sweepings, 0.071 (allowing 33% as incombustible); making a total of 0.295 lb.

If the heat is passed through a boiler, and the losses be assumed at 40%, then the available useful heat would be 2 391 $\times$ 0.60 = 1 434 B. t. u., or sufficient to evaporate 1.48 lb. of water from and at 212° fahr.

If 1 lb. of coal can evaporate 9 lb. of water, and is worth $3 per

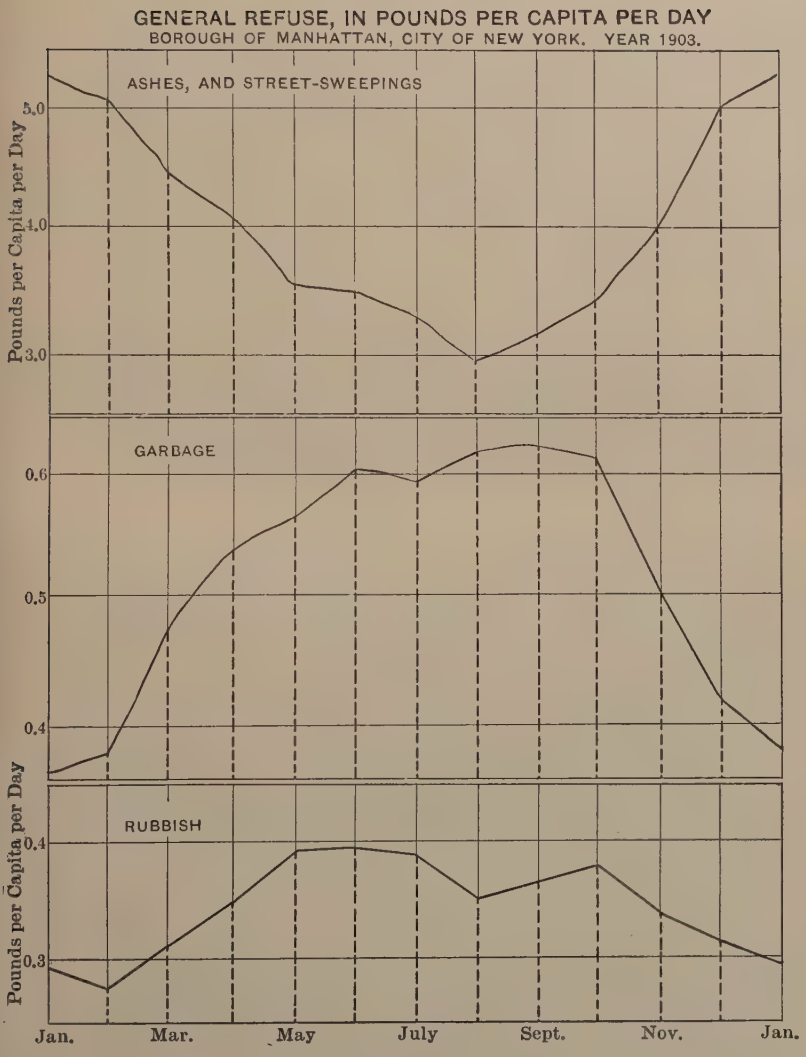

GENERAL REFUSE, IN POUNDS PER CAPITA PER DAY
BOROUGH OF MANHATTAN, CITY OF NEW YORK. YEAR 1903.
ASHES, AND STREET-SWEEPINGS
Pounds per Capita per Day
5.0
4.0
3.0
GARBAGE
0.6
0.5
0.4
RUBBISH
Pounds per Capita per Day
0.4
0.3
Jan.
Mar.
May
July
Sept.
Nov.
Jan.

FIG. 5.

ton, then the value of this refuse as a fuel would be ($1.48 ÷ 9) × 3.00 = $0.49 per ton.

TABLE 12.—Estimate for Heat Balance.

Temperature of Air, 60° fahr. Temperature of Furnace, 2 000° fahr. Fuel: City Refuse, Combined Collections—One Pound, moderately dry.

		Total heat of combustion. B. t. u.	Dissipation of heat. B. t. u.
Garbage:	Dried portion; 0.229 × 0.30 × 7 500	515	
	Moisture; 0.229 × 0.70 × [(212—60) + 966 + 0.48 (2 000 — 212)]		317
Ashes:	On the basis that only three-fourths of the combustible is burned; then 0.487 × 0.75 × 3 000	1 095	
Rubbish:	On the basis that one-third has been sorted out; then 0.071 × 5 000	355	
Street-sweepings:	On the basis that there is 33⅓ incombustible matter; then 0.213 × 2 000	426	
Heating fuel mass:	Taking the specific heat of the mass at 0.2; then 1 × 0.2 × (2 000 — 60) = 388, of which, say, one-third is lost through raking out the hot ashes		129
Heating air supply:*	On the basis of 4.2 lb. of air per lb. of fuel; then, 4.2 × 0.2375 (2 000 — 60). This is about 14.3 lb. of air per lb. of combustible.		1 945
	Estimated British thermal units per pound	2 391	2 391

* Pounds of air per pound of fuel calculated by difference.

It would be possible to vary the proportions of the constituents fed to the furnace, as long as sufficient combustible material was supplied to evaporate the moisture in the garbage. This condition can be attained by practice at the furnace. It is also self-evident that an increased efficiency will be obtained if the air supplied be heated.

In the same manner, another heat balance can be worked out for the burning of garbage, rubbish and street-sweepings, when all the ashes are kept out of the furnace and utilized for land filling.

Taking the same general analysis as before for the combined collections, the fuel proportions would be as shown in Table 13.

Assume as before that one-third of the rubbish is sorted out as marketable; then the percentage of the constituents as thrown into the furnace would be in the ratio stated in the last column of Table 13.

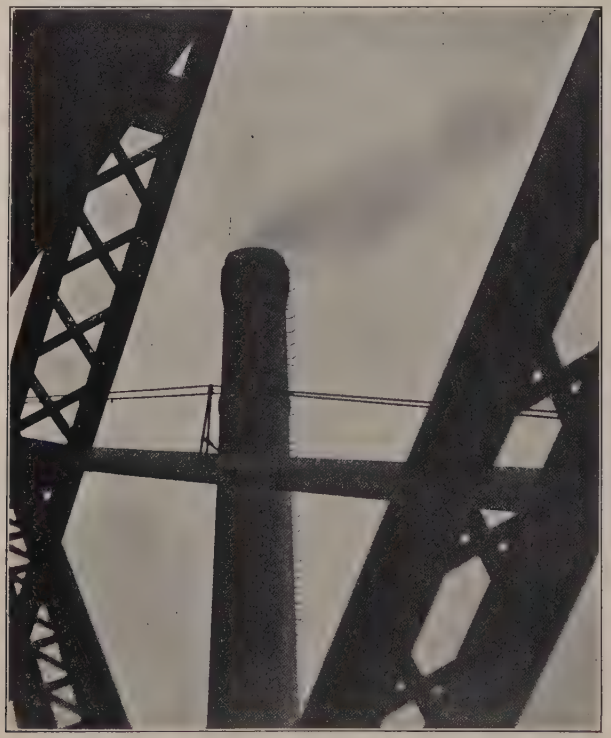


FIG 1.—SMOKE FROM RUBBISH INCINERATOR, WHEN BURNING DRY RUBBISH.

FIG. 2.—SMOKE FROM RUBBISH INCINERATOR, WHEN BURNING WET RUBBISH.

TABLE 13.

Classification.	City collections.	Used as fuel.
Garbage	15% (70% moisture)	44.5%
Rubbish	7% (33⅓% sorted out)	13.9%
Street-sweepings	14%	41.6%
		100.0%

TABLE 14.—Estimate for Heat Balance.

Temperature of Air, 60° fahr. Temperature of Furnace, 2 000° fahr. Fuel: City Refuse—Garbage, Rubbish and Street-Sweepings, One Pound.

	Total heat of combustion. B. t. u.	Dissipation of heat. B. t. u.
Garbage: Dried portion; $0.445 \times 0.30 \times 7\,500$	1 001	.....
Moisture; $0.445 \times 0.70 \times [(212 - 60) + 966 + 0.48\,(2\,000 - 212)]$	.....	615
Rubbish: $0.139 \times 5\,000$	695	.....
Street sweepings: $0.416 \times 2\,000$	832	.....
Heating fuel mass: $1 \times 0.2 \times (2\,000 - 60)$, of which about one-third is lost	.....	129
Heating air supply: On the basis of 8.9 lb. of air per lb. of fuel; $8.9 \times 0.2375 \times (2\,000 - 60)$. This is about 9.9 lb. of air per lb. of combustible	.....	1 784
Estimated British thermal units per pound	2 528	2 528

The combustible portion is: garbage, 0.1335; rubbish, 0.1251 (allowing 10% for the incombustible portion); and street-sweepings, 0.1386 (allowing 33% as incombustible); making a total of 0.3972 lb.

If the heat is passed through a boiler, and the losses are assumed at 40%, then the available useful heat would be $2\,528 \times 0.60 = 1\,516$ B. t. u., or sufficient to evaporate 1.57 lb. of water from and at 212° fahr.

On the basis of 34.5 lb. of water evaporated from and at 212° fahr., as equivalent to 1 b. h. p., these two results would represent a boiler horse-power for about 23.3 lb. of combined collections and 22.0 lb. of mixed garbage, rubbish and street-sweepings.

The results also indicate that, in order to obtain a high furnace temperature, an artificial draft is needed, so as to get the benefit of a thorough mixture of the oxygen with the fuel and not require too great a surplus of air. In practice, the highest evaporative results per pound of collection burned have been obtained when the boiler is set directly over the fires, as the heat is direct and the losses are reduced to a minimum. Such a plan, however, is not conducive to the maintenance of high temperatures in the furnace, and, consequently, the complete destruction of the mass by thorough combustion is apt to be sacrificed. As the latter consideration is of paramount importance, the boilers should be set to receive the gases of combustion after the combustion has been completed, and while still at high temperature.

The experience obtained in England by the burning of mixed collections shows an evaporation varying from ½ to 2 lb. of water from and at 212° fahr. per lb. of refuse. The best results are obtained by the use of artificial drafts, so as to obtain the benefit of a thorough mixture of the gases with the smallest air supply. The air supplied in the blasts is about 3½ lb.* per lb. of average mixed refuse.

Table 15 gives some figures relative to the total heats of combustion of the constituent elements of refuse.

TABLE 15.—Approximate Calorific Values of Refuse.

	British thermal units. per pound.	Authority.
Garbage, dry	7 500	H. de B. Parsons.‡
" as collected	800	" ‡
Bones and offal, dry	8 000	Dawson.†
" " " average moisture	5 333	"
Ashes, combustible portion	12 000	H. de B. Parsons.
" as collected, average	3 000	"
Rubbish, from	7 500	" ‡
to	5 000	" ‡
Paper, straw, fibrous matter, and vegetable refuse, dry	3 800§	Dawson.†
Paper, straw, fibrous matter, and vegetable refuse, average moisture	2 500§	"
Rags, dry	5 000	"
" average moisture	3 333	"
Wood, dry	7 800	H. de B. Parsons.
" average moisture	6 500	"

* George Watson. *Transactions*, Am. Soc. M. E., Vol. XXV, 1904.

† "Disposal of Municipal Refuse," *Transactions*, Am. Soc. C. E., Vol. LIV, Part E, 1905.

‡ Estimated from its composition, and verified by its behavior in the incinerators.

§ The writer considers this too small.

FIG. 1.—FRONT OF EAST FURNACE, SHOWING STOKING DOORS.

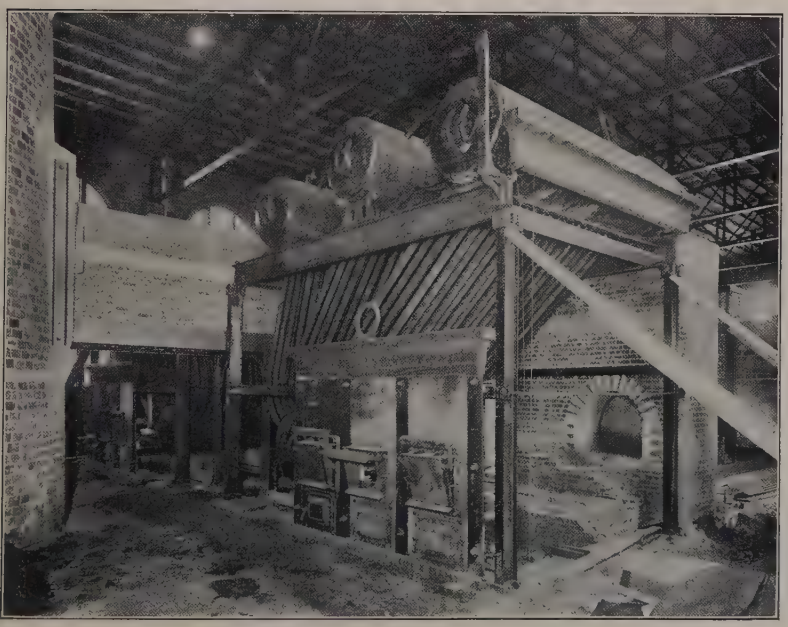


FIG. 2.—BOILERS FOR WILLIAMSBURG BRIDGE LIGHTING STATION AND RUBBISH INCINERATOR PLANT.

The Rubbish Incinerator.

In New York the city refuse is separated into classes, and collected by different carts. The rubbish collections include paper of all kinds, books, magazines, cardboard, rags, barrels, boxes, crates, shoes, hats, pieces of leather, rubber, cans, metals, garments, beds, bedding, mattresses, bed springs, sofas, chairs, broken furniture of all kinds, and various other articles, both large and small, of almost every conceivable nature. The material is heterogeneous and bulky. As packed in the collection carts, its weight averages 141 lb. per cu. yd., which is the average of the actual weighing of 135 carts from different districts.

As this material makes a very poor land-fill, and as it cannot be dumped at sea without danger of its floating back on the beaches, the rubbish incinerator plant was designed by the writer for the Department of Street Cleaning, in order to reduce its bulk, to transform it into ash which can be used for land-fill, to provide better picking facilities than is afforded by dumping directly into scows, Fig. 1, Plate XI, and to destroy in a sanitary manner the beds, bedding, mattresses and furniture, some of which may be carriers of disease.

Location.—The plant is located on city property, beneath the Williamsburg Bridge, facing Delancey Slip, between East Street and Tompkins Street, Borough of Manhattan. It is thus close to the East River, being separated therefrom only by the width of East Street.

General Arrangement.—The arrangement of the plant is shown on Plate XII. The collection carts drive into the building through any of the four doors on the Delancey Slip front, and dump their loads on the belt conveyor. If the material arrives faster than it is wanted on the conveyor, the loads are dumped on the floor of the receiving room, the material then being pushed upon the conveyor by hand labor as desired. The empty carts go back for more material.

The conveyor carries the material between two rows of "trimmers," as the men are called who pick out the marketable material and place the different kinds in separate boxes. This marketable material is baled by presses and by hand, and removed by the contractor, who pays the city for the privilege.

The material not picked passes to the end of the conveyor, where it is dumped on a steel charging floor, and pushed into the furnaces by furnace men.

The street-sweeping carts drive up an inclined runway and dump directly on the charging floor, thus preventing the street dirt from injuring the salable portion of the rubbish.

The furnaces are connected with a stack, and, by means of dampers, the hot gases may be diverted either directly to the stack or through the boilers.

On one side of the building doors are provided for carts to enter to remove the ashes and the baled material without interfering with the material being received.

A second floor over part of the building provides storage space for the material to be burned on holidays when there is no collection, and for night use. The receiving floor also provides additional space for storage, as well as the yards on both sides of the building, and a scow, filled at some other dumping station, is, furthermore, kept at the wharf opposite the building. In this way there is no trouble from lack of material to burn.

Building.—The building is 150 ft. deep and 70 ft. wide. The ground floor is at the level of the sidewalk, and is paved with Belgian blocks. Over the northern half and along the west side there is a second floor. This upper floor, used for storage, is reached by an inclined driveway outside the structure.

The front of the building, Fig. 2, Plate XI, is of brick, with buff brick facing, and the sides are of Phœnix, hollow, tile-block construction, stuccoed on the outside. The pilasters beneath the roof trusses are of red brick, locked into the tile blocks, course and course.

The roof is supported by steel trusses and purlins, and is covered with saturated roofing felt, coal-tar pitch and gravel. The upper floor is of reinforced concrete on steel beams and girders, supported on steel pipe columns.

The windows are as large as could conveniently be made, consistent with strength in the walls.

A division wall of tile blocks, carried to the upper floor, divides the receiving space from the furnace space, and keeps the combustible material away from the fires. This wall does not reach en-

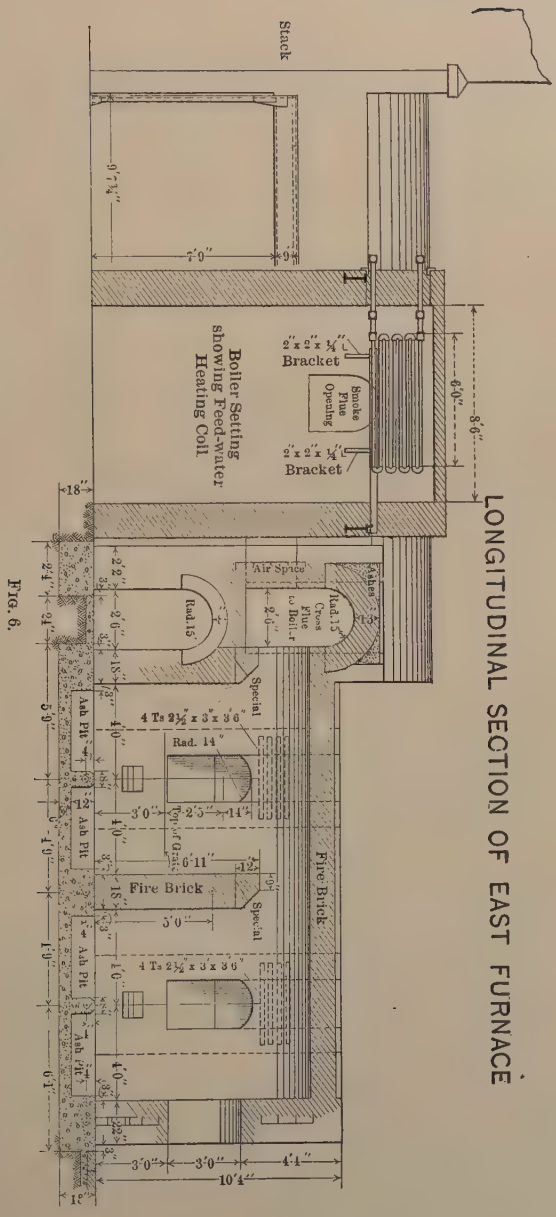

Stack
Boiler Setting showing Feed-water Heating Coil
2″ x 2″ x ¼″ Bracket
Smoke Flue Opening
2″ x 2″ x ¼″ Bracket
8′6″
6′0″
Air Space
Ashes
Rad. 15
Cross Flue to Boiler
Special
4 Ts 2½″ x 3″ x 3′6″
Rad. 14″
Fire Brick
Ash Pit
LONGITUDINAL SECTION OF EAST FURNACE

FIG. 6.

tirely across the building, and the space is used as a driveway for ash carts, which can enter at one of the side doors and pass out at the other.

Conveyor.—The conveyor is of the metallic, apron type, 48 in. wide, with side angles to hold the rubbish. The flights are each 6 in. long and the full width of the conveyor, and are lapped over each other, so that there is no space between them as they travel over the head shafts. Fig. 1, Plate XIII, shows the conveyor as it rises from the receiving floor to the tops of the furnaces, and passes between the sorting boxes.

By spreading out the material on the conveyor the opportunity for picking is greatly facilitated. The picking and sorting privilege is let to a contractor, who pays a sum sufficient to cover more than all the labor charges in the plant. The amount of material picked out varies with the market value for the old paper, cans, etc., between 25 and 40% by weight of the material delivered. Some of this material, baled and ready for removal, is shown in Fig. 3, Plate XI.

To facilitate the unloading of scows, which bring rubbish from other city districts, a conveyor across East Street has been designed, but not yet constructed, Plate XIV. This will unload the material on the upper floor, from which it will be pushed through a chute to the main conveyor.

Both these conveyors are arranged for power drives, using steam from the boilers. The speed of the conveyor is controllable, but the working speed is about 50 ft. per min.

Charging Platform.—The platform over the furnaces is of steel construction, covered with steel floor-plates. The conveyor dumps upon this platform, Fig. 2, Plate XIII.

Furnaces.—There are two furnaces, of somewhat different design. The west furnace was designed by F. L. Stearns, Assoc. M. Am. Soc. C. E., and has two grates for part of its length. The area of the upper grate is 113 sq. ft. The east furnace was designed by the writer, and has one grate, 74 sq. ft. in area, divided into two cells by a division wall, as shown in Fig. 6. The material is fed through two chutes at the back, is stoked forward on the grate as wanted, and the clinkers are pulled out through the stoking doors, Fig. 7. The motion of the material is thus continuous. This sys-

tem has worked well, with the exception of the damage to the fire-brick lining due to the stoking tools, and improvements are planned to remedy this trouble.

Both furnaces are of brick, with fire-brick lining, and are strongly tied with stays made of 8-in. channels set in pairs.

The hot gases pass through a cross-flue, with dampers, either to the boilers or to the stack. The dampers are of special tile, and necessarily very large, Fig. 8. They are raised and lowered by triplex blocks and chains.

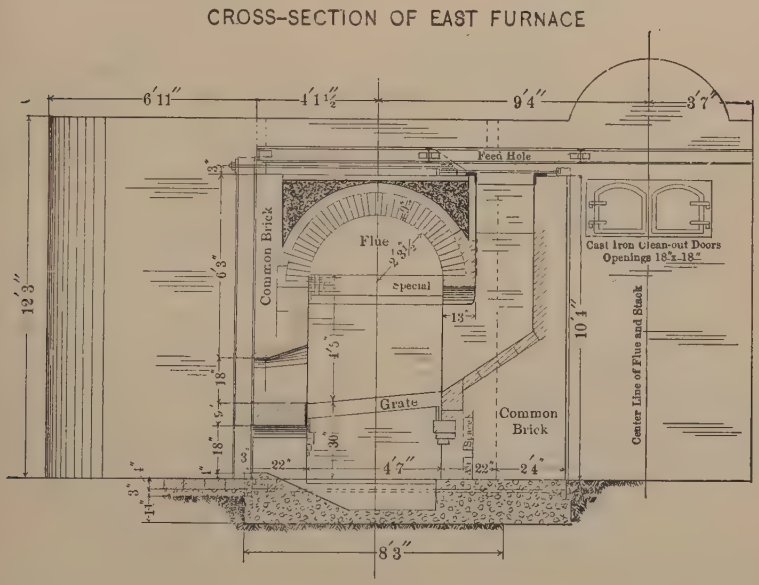
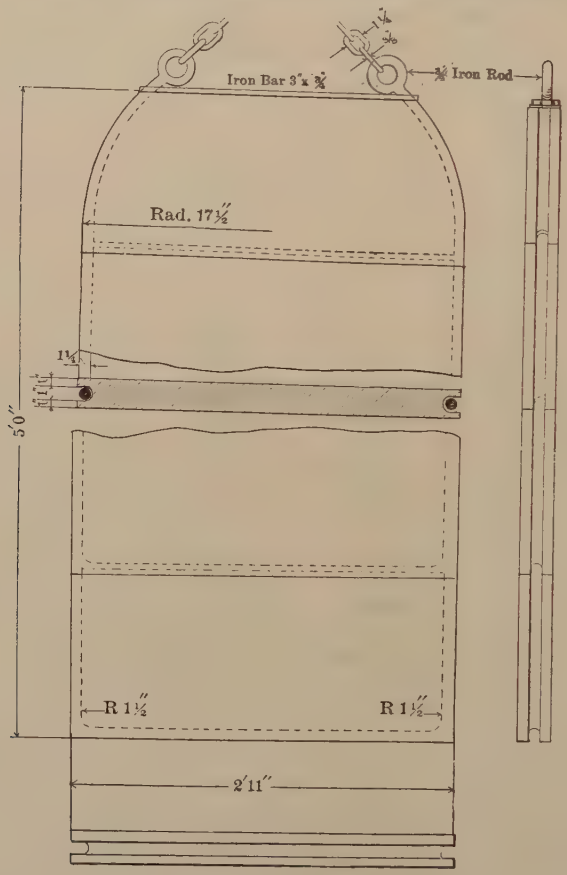


FIG. 7.

The material burns freely, often consuming at the rate of 40 lb. per sq. ft. of grate per hour. Even when wet, that is, rainy day collections, or when wet street-sweepings are fed in, the effect is not detrimental to rapid combustion. When dry, the smoke from the stack has a light bluish color, and when wet a light yellowish tinge. Rarely does the smoke exceed the amount shown in Figs. 1 and 2, Plate XV. These photographs were taken when the furnaces were burning about 35 lb. per sq. ft. of grate per hour, the former when the rubbish was dry and the latter when it was wet.

On the end of each furnace there is a large door, to feed in beds, furniture and similar bulky articles. Fig. 1, Plate XVI, shows the stoking doors on the front of the east furnace.

Boilers.—The boilers are Stirling water tubular, Fig. 2, Plate XVI, and each contains 1 890 sq. ft. of effective heating surface.



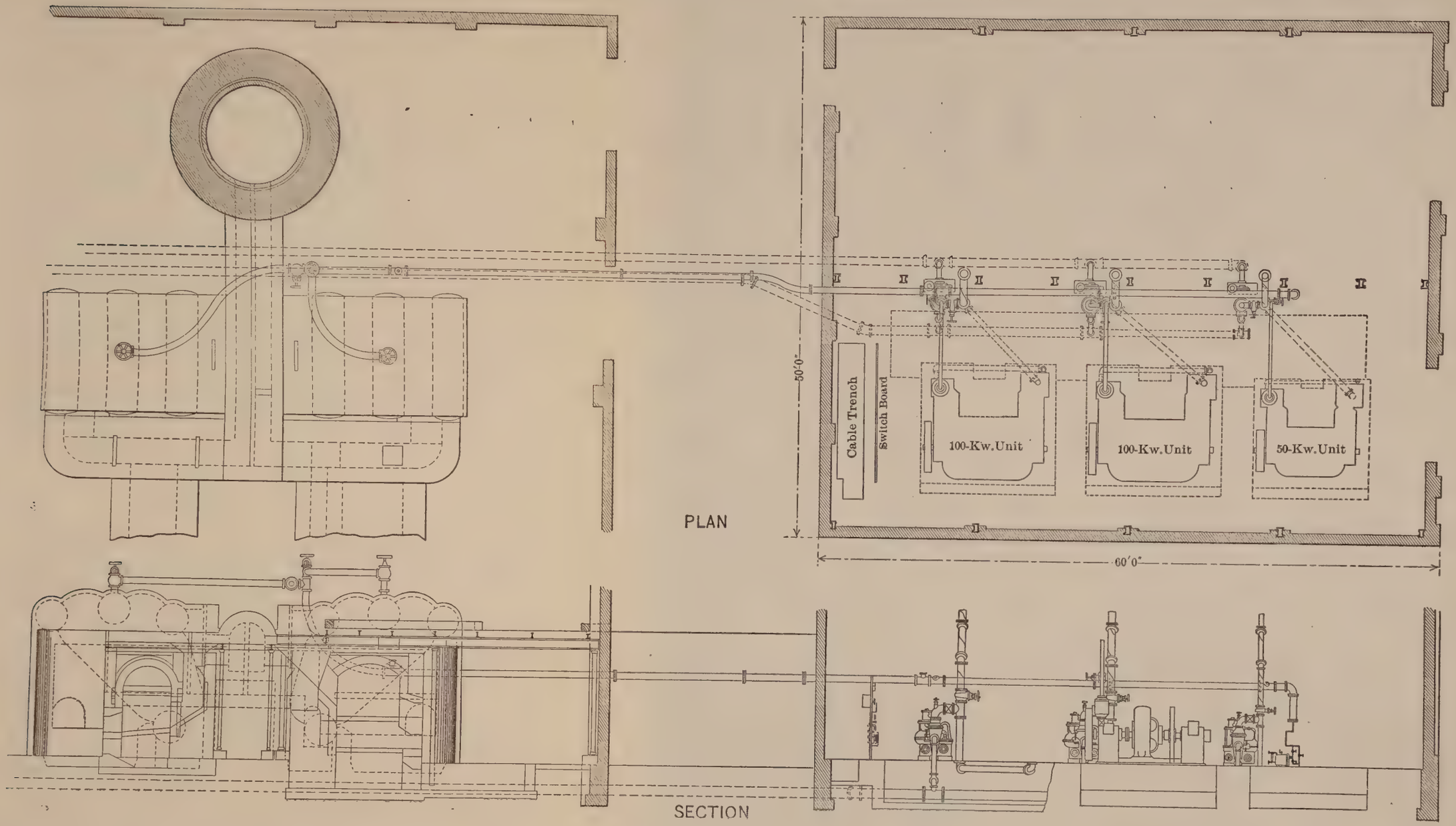
TILE DAMPER USED IN FLUE

Fig. 8.

The tubes are 3¼ in. in diameter. Each boiler has its regular coal grate, and as the hot gases from the rubbish furnaces enter above these grates, they are always ready for coal firing (by simply arrang-

PLATE XVII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVII, No. 1027.
PARSONS ON
REFUSE DISPOSAL.

PLAN AND SECTION THROUGH ELECTRIC LIGHTING STATION.



ing the dampers), should any accident happen to the fuel supply. The coal firing doors are placed on one side of the boiler setting, instead of at the end, as usual, a change made to accommodate the arrangement of the plant.

Each boiler has a feed-water economizer coil of brass pipe, set in the flue to the stack. These coils are connected with unions and can be withdrawn at any time without stopping the plant.

Stack.—The stack is of hollow, radial-block construction, built by M. W. Kellogg and Company. It is 200 ft. high, so that the top would be 75 ft. above the roadway of the bridge. The outside diameter at the base is 17 ft. and at the top 5 ft. 9 in. The stack is lined to a height of 135 ft. above the foundation. The lining above the smoke inlet is of unusual construction, being in the form of a continuous helical band, 16 ft. 5 in. high at any point. It is supported on corbels, but, being arranged helically, there are only three corbel blocks in each horizontal row of the outer column. The object of this helical arrangement is to prevent the outer column from cracking, when exposed to sudden changes in temperature, due to continuous corbeling in horizontal courses. This arrangement is shown in Fig. 9.

The foundation is of concrete, supported on piles.

The Electric Lighting Station.

Building.—In order to utilize the steam generated, an electric lighting station was built by the Department of Bridges, from plans by the writer. The building is 60 ft. deep and 50 ft. wide, and is separated from the incinerator building by 20 ft.

It is of steel skeleton construction, with the trusses supporting a reinforced cinder-concrete roof, covered with felt and coal-tar pitch and gravel. The curtain walls are of brick, with large windows, and are made to match the other building.

The north half of the building has a second floor of reinforced cinder-concrete construction, designed to be used at some future time for a storage battery plant. The general arrangement is shown in Plates XII and XVII.

Machinery.—There are two 100-k.w. and one 50-k.w. direct-connected units. The generators are multipolar, direct-current ma-

chines, built by the Burke Electric Company, and arranged for the three-wire system. They are all wound for 250 volts, with 125 volts on either side.

The generators are driven by "Ideal," cross-compound, horizontal, condensing engines, the large units having cylinders 12 and 20 in. in diameter with 12-in. stroke, and the small unit 9 and 16 in. in diameter with 10-in. stroke.

Each unit has an independent Blake, vertical beam, jet condenser, for which the suction and discharge are connected to the East River. The exhausts are all by-passed, and extend through the roof, so that the engines can be operated as non-condensing.

The exhausts from the condensers and feed pumps are passed into an open feed-water heater of the "Cockrane" type, into which are also returned the discharges from all traps that do not handle oil or grease.

Piping.—The piping is as simple as possible. The steam main is 7 in. in diameter, graded down from the boilers, with the far end turned down to form a pocket for drainage of condensation, and trapped. The supply branches are all taken from the top, and expansion is provided for by easy bends.

Current.—The electricity generated is used to light the Williamsburg Bridge, the incinerator building and the lighting station.

The cables are in underground conduits to the foot of the intermediate tower of the bridge, whence they are carried up the tower to the electric distribution of the bridge.

The number of lights, etc., at present connected, is given in Table 16, but when the storage plant is completed, the electric station can have a day load, and the current thus stored can be used for additional work as well as to assist the bridge load in cases of accident.

TABLE 16.—Lights, etc., Supplied by Lighting Station.

Lights, etc.	Bridge.	Incinerator.	Station.
2 000-c. p. arc lamps	168	8	4
16-c. p. incandescent lamps	707	40	20
Electric heaters	20	.....	.....
Electric motors	3	.....	.....

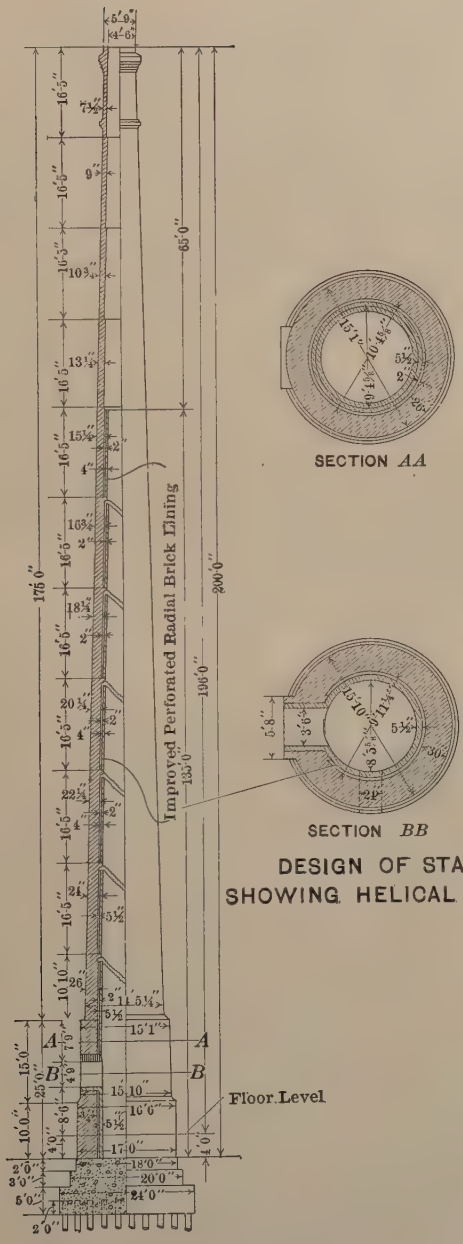

Improved Perforated Radial Brick Lining
Floor Level
SECTION AA
SECTION BB
DESIGN OF STACK,
SHOWING HELICAL LINING.

FIG 9.

The plant was formally opened by Mayor McClellan on October 30th, 1905. The cost of the plant was as follows:

Incinerator, including building, stack, east furnace, and equipment	$26 768
West furnace	4 000
Runway	1 550
Conveyor	1 875
Lighting station, including building, all machinery, boilers, and electrical equipment.	49 391
Total cost	$83 584

On December 20th and 21st, 1905, the writer conducted evaporative tests, the data and result of which are given in Table 17. Each boiler, with its furnace, was tested separately, under similar conditions.

Figs. 1 and 2, Plate XV, show the average condition of the smoke during the tests.

Fig. 10 shows an average steam card produced on a night run.

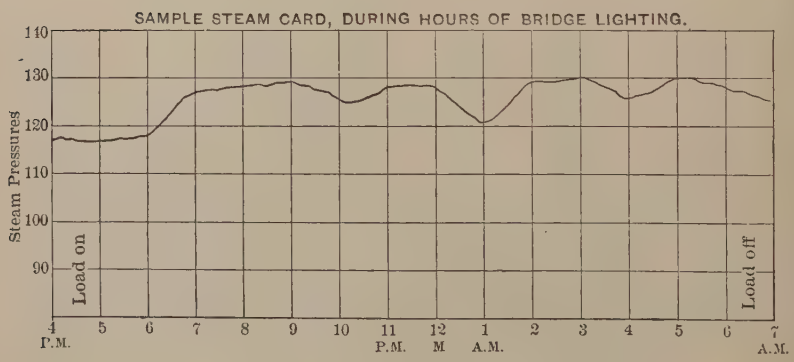


Fig. 10.

During the ordinary weekday, the quantity of rubbish consumed is about 160 loads, equivalent to 22 560 lb.

TABLE 17.—Data and Results of Evaporative Tests; Rubbish Incinerator and Electric Lighting Station, Delancey Slip, Borough of Manhattan, New York.

Trials made by H. de B. Parsons.
Kind of fuel, city rubbish collections.
Method of starting and stopping, alternate.
Make of boilers, Stirling Water Tubular.

Data.	West boiler.		East boiler.	
Grate surface of furnace		113 sq. ft.		74 sq. ft.
Effective water-heating surface		1 890 " "		1 890 " "
Surface of feed-water heater coil in flue		60 " "		60 " "
TOTAL QUANTITIES.				
Date of trial		Dec. 20th, '05.		Dec. 21st, '05.
Duration of trial		5.5 hours.		5.5 hours.
Weather		Fair.		Rainy.
Condition of rubbish		Dry.		Wet.
Weight of rubbish delivered		31 193 lb.		21 175 lb.
Weight of rubbish picked out as:				
marketable		8 926 "		7 245 "
paper	6 876		6 435	
rags	1 800		610	
cans	250		200	
Weight of rubbish burned		22 267 "		13 930 "
Weight of ash, estimated		10%		10%
Total weight of water fed to boiler		29 925 "		24 675 "
Equivalent water evaporated, from and at 212°		36 568 "		30 054 "
Number of furnace men:		9		4
stokers	6		2	
feeders	3		2	
Equivalent evaporation per man per ton		365 "		1 078 "
HOURLY QUANTITIES.				
Rubbish consumed per hour		4 048.5 "		2 532.7 "
" " " " per square foot of grate		35.8 "		34.2 "
Water evaporated per hour		5 440.9 "		4 486.2 "
Equivalent evaporation per hour, from and at 212°		6 648.7 "		5 464.2 "
Equivalent evaporation per hour, from and at 212°, per square foot of heating surface		3.51 "		2.89 "
AVERAGES.				
Temperature of external air		46° fahr.		49° fahr.
Barometer, inches		30.32		29.64
Steam pressure by gauge		117 lb.		100 lb.
Temperature of feed-water		40° fahr.		40° fahr.
Temperature of gases entering boiler				1 740° "
" " " under "		1 525° fahr.		1 400° "
" " " escaping from boiler		466° "		412° "
Force of draft, under boiler, inches		0.75		0.55
" " " in flue to stack, inches		1.30		1.15
Boiler horse-power developed		192.7		158.4
ECONOMIC RESULTS.				
Water evaporated, actual, per pound of rubbish		1.34 lb.		1.77 lb.
Equivalent evaporated per pound of rubbish		1.64 "		2.16 "

The cost of operating the plant by the City, showing the debits and credits, is given in the financial statement, Table 18. Prior to the construction of the plant, the material was loaded on scows and taken to land-fills. (Previous to the administration of Commis-

sioner Woodbury, much of this material was dumped at sea.) These scow loads contained the collections of ashes, street-sweepings and rubbish. The rubbish, therefore, was reduced in bulk, both by compression and by the ashes filling in the voids.

The cost to the City, for towing, unloading and scow hire, averaged, in 1905, $0.1569 per cu. yd. of mixed material. From a study of the records kept by the City, the reduction of the rubbish as collected and delivered occupies three-tenths of its original volume when loaded on the scows.

An average day's work at the plant represents about 1 050 cu. yd. of rubbish delivered during 24 hours.

TABLE 18.—Financial Statement.

One Day's Work. Delancey Slip Plant.

Cost of disposal of rubbish on land-fills.		
1 050 cu. yd. delivered, compressed on scows, after trimming, to 315 cu. yd.		
315 cu. yd. at $0.1569		$49.42
Cost, Incinerator.		
Labor	....	
Ash removal, 7.6 cu. yd. at $0.1569	$1.20	
Supplies and repairs	8.00	
Interest, 3½% on $34 193	3.28	
		12.48
Saving per day		$36.94
Saving per year, $13 483, or 39.4% on cost.		
Cost, Electric Lighting Station.		
Cost of buying electricity		$80.00
Labor	$20.00	
Supplies, repairs and sundries	8.00	
Interest, 3½% on $49 391	4.74	
		32.74
Saving per day		$47.26
Saving per year, $17 250, or 35% on cost.		

The total saving, as shown by Table 18, on the combined plant is $30 733 per annum, or 36.7% on the cost.

There is included in Table 18 no cost for labor charges in the incinerator plant, because the privilege of picking out the marketable rubbish on the belt conveyor is under contract, and the contractor pays to the City a sum which slightly exceeds the expenses of labor in the incinerator building, including the operation of the boilers. Taxes are not included, because the plant is built on City property, purchased to protect the bridge structure overhead.

Prior to the erection of the Delancey Slip plant, the Department of Street Cleaning made tests on the burning of rubbish in a building on North Moore Street and at the rubbish incinerating plant erected at the foot of West Fourth Street, with the results given in Table 19. These experiments were carried on during 1903 and 1904. At the North Moore Street plant a second-hand return-tubular boiler was used, with a grate and setting arranged to accommodate the rubbish.

TABLE 19.

	North Moore St.	Forty-seventh Street Incinerator.	
Duration of trial, hours	6	2	3
Grate surface, square feet	.....	90	90
Boiler heating surface, square feet	324	2 760	2 760
Rubbish burned, pounds	3 324	9 3 6	10 054
Temperature of feed water, degrees, fahr.	56°	50°	50°
Steam pressure, pounds	10	80	80
Water evaporated, pounds	3 968.75	11 101.00	.....
Equivalent evaporation, from and at 212° fahr., pounds	4 648	13 365	15 139
Equivalent evaporation, from and at 212° fahr., pounds per pound of rubbish	1.40	1.43	1.50
Temperature of furnace, degrees, fahr.	1 500°	.....	.....
Ashes removed, pounds	.....	907	.....
British thermal units required for evaporation of water from and at 212° fahr., per pound of rubbish	1 352.4	1 381.4	1 449.0

The ashes produced at the Forty-seventh Street and at the Delancey Slip rubbish incinerating plants were analyzed by the Lederle Laboratories, with the following results:

Sample of Ashes from West Forty-seventh Street Incinerator:

Moisture	2.12%
Potassium carbonate	2.65%
Calcium phosphate	1.98%
Alkaline earth carbonates, silicates, soda, oxides of iron and alumina, etc.	68.05%
Organic and volatile matter (loss on ignition)	25.20%
	100.00%

Sample of Ashes from Delancey Slip Incinerator:

Moisture	0.75%
Nails and other metal	5.48%
Broken glass	4.05%
Bone phosphate	2.71%
Potash	0.46%
Alkaline earth carbonates, silicates, soda, oxides of iron and alumina, etc.	60.91%
Organic and volatile matter (loss on ignition)	25.64%
	100.00%

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1028.

ENGINEERING PATENTS.

An Informal Discussion at the Annual Convention, June 27th, 1906.

SUBJECT FOR DISCUSSION.

"THE PROTECTION OF THE INTELLECTUAL PROPERTY OF CIVIL ENGINEERS."

BY MESSRS. D. A. USINA AND J. P. A. MAIGNEN.

D. A. USINA, ASSOC. AM. SOC. C. E. (by letter).—The application of the patent laws to the work of civil engineers presents certain interesting peculiarities; and, in connection with these, is to be considered the question of protecting such intellectual products of the engineer as are not reached by the law. Mr. Usina.

Civil engineers are here spoken of in the more restricted sense; not including mechanical or electrical engineers, chemists, or metallurgists, or others whose works may be multiplied many times without substantial changes; but referring more particularly to the builders of great hydraulic works, systems of water distribution and sewerage, great dams, railway systems, bridges, and buildings in steel or masonry. Works of the latter class cannot be exactly duplicated. Each dock or harbor improvement presents fresh problems. Though the details of railways, bridges, and buildings, and even certain large features of design, have become more or less standardized, yet there is not the capability of duplication regardless of conditions which exists in mechanical or electrical machines or chemical products or processes.

Mr. Usina. Each work of this sort calls for originality. A scheme for tunneling the East River might be impracticable for the Hudson River; or a scheme for tunneling the Hudson might involve expenses entirely unnecessary in other situations. A civil engineer is called upon to display originality in each new work which arises. The "skill of the profession" involves, not only fundamental knowledge, but an ability to apply it to constantly varying situations. To a much less degree is this originality required in other professions.

But this originality, this inventive quality, is not as well rewarded by our patent laws as the same degree of originality in designing new machines. The design of a new pumping engine, for example, may be multiplied a thousand times, while a new tunnel, involving a greater degree of invention, will be capable of use in but one or two locations. During seventeen years the inventor of the pump is receiving royalties based upon the value of his one improvement. The tunnel engineer, on the other hand, because his work is adapted only to one situation, receives but a single payment—the usual engineer's fee.

There are, of course, some circumstances under which civil engineers have found it advantageous to protect by patent an invention which was capable of use in only one location. For example, the intention of the Pennsylvania Railroad to build a tunnel under the Hudson River was so well advertised, and the peculiar difficulties so thoroughly discussed in the technical journals, long before the work was let, that many engineers conceived original and ingenious solutions of the difficulties; and, in order that they alone might profit by their ingenuity, secured patents for the processes proposed. Speaking from recollection, there were more than a dozen such patents issued at about the time the work was to begin. Such a patent, of course, may be highly remunerative if it proposes a plan which is clearly the best, and if it secures for the patentee the contract to build the work, or a royalty for the use of his idea; but it is not likely to be of value unless adopted on this particular work.

The civil engineer, however, ordinarily includes his inventive ability in the services for which he is retained, and receives no reward for a good design except a "well done" from the few who can appreciate the difficulties which he has surmounted. Out of this fact undoubtedly springs the skeptical attitude of many civil engineers toward patents. One of our best known bridge engineers recently said, of a patented bridge:

"Why, there's no room for a patent there! It's merely a nice application of known engineering principles to a peculiar set of circumstances."

Yet the design of the patented bridge was certainly new, and it is not safe to say that any good engineer, given the same problem, would have evolved the same design. In fact, the range of possible solutions is so great as to make it improbable that any two engineers would have arrived at the same solution of the problem; and experience shows that they seldom do arrive at the same solution. Is it not fair, then, to give the inventor the benefit of so much as is original with him? This is all that the patent law aims to do. New principles cannot be patented, nor obvious applications of old principles; but, in exchange for his giving to the world the benefit of his original inventive ability, the law purposes to give the inventor a monopoly of his creation for a limited term. Mr. Usina.

The distinction which the law makes against devices the conception of which involves only the expected skill of the profession, is illustrated by a case which came under the writer's personal observation some years ago. An application was filed for a patent for a railway system, including express tracks and local tracks, with outside local stations and "island" express stations at which passengers could transfer from local to express trains, such as now used in the Manhattan subway. The advantages were set forth in detail, and could not be denied. Nor could any previous record of such a system be found. Yet the very liberal Examiner of the Civil Engineering Division held that a good engineer, given conditions favoring the utilization of such a design, would naturally evolve it; and the patent was properly refused.

In reference to the law, the word "purposes" is used advisedly. The work of law-makers is no more perfect than that of engineers; and the execution of the law by men introduces another fallible element. What is an "obvious" application of known principles, as distinguished from a true invention involving more than the expected skill of the profession? There are thirty-eight examiners in the Patent Office, and each has his own standard of what constitutes invention. By a wise provision of the law, they are expected to give applicants the benefit of any doubt, but, however the standard may be stated in words, there will be as many interpretations of it as there are examiners. The tendency of many examiners, and especially of those newly appointed, is to regard applications for patents as little better than attempts to obtain monopolies to which the applicants are not entitled, and this tendency results in putting upon applicants the often heavy burden of proving that they are entitled to a patent.

Civil engineers have been fortunate in that they have had for many years at the head of the Division of Civil Engineering an examiner with broad ideas, who has looked upon his work as not merely obstructive. But the writer has known applications, par-

Mr. Usina. tially allowed by this examiner, to be transferred to another examiner recently appointed, and to be by him rejected *in toto;* and not because of the discovery of any new grounds of rejection, but because of a narrower view and less familiarity with the art in question, and probably because of a fear of taking the responsibility of allowing a patent the validity of which was not absolutely certain. The law contemplates the allowance of a patent unless it is clearly invalid, but this examiner refuses it unless its validity is practically certain. There are appeals, of course, but they are expensive, and the attitude of the primary examiner determines in a large degree the nature of the patents issuing from his particular division of the Patent Office.

The ethical proposition has been advanced that it is the duty of the civil engineer to give to the public the benefit of his invention; that his function is to advance his art and to assist his brethren in doing the same thing; in other words, to give to all engineers freedom to use the methods and tools which his brain has created. And this is not to be denied. In only one other profession—the medical—is the same philanthropic view held.

But this view is not inconsistent with a monopoly for a limited period of time. Seventeen years, the term of a patent, is not a very long period in the progress of an art, and the ultimate advance is not retarded by such a monopoly. On the other hand, the reward which may be gathered in this limited term is a stimulus to invention, and a just compensation for the study and the often heavy expense needed to reduce a mere idea to a practical and useful embodiment. The legal theory of a patent is that it is a contract in which each party gives an equivalent. The Government gives a limited monopoly; the inventor gives to the world a complete description such as will enable others to use his invention when the monopoly expires. And patents have been held invalid for lack of such a full, clear and exact description as will put the invention at the disposal of the public. The consideration for the monopoly having been withheld by the inventor, the monopoly has also been withheld.

The large works of civil engineers are seldom duplicated in a period of seventeen years, and the rewards, in the shape of royalties, upon patents for such works, are not numerous; but that is rather a reason for granting than for denying the patent. There is very little danger of imposing on the public. The courts give to an infringer of a patent ample opportunity for showing, either by publications or by actual works, that the patentee was not the original and first inventor of the matter in question. And though the grant of a patent by the Patent Office carries with it a presumption of validity, yet this presumption may be easily over-

thrown if it is not justified in fact. The law requires that the patentee shall give a full, clear and exact disclosure of the invention, that he shall be the true inventor, and that the conception shall involve more than the expected skill of the calling. Under these circumstances, justice certainly calls for a reward proportional to the benefit conferred upon the subsequent users of the invention. Mr. Usina.

This reward is noticeably a great stimulus to invention (and the extreme liberality of our patent laws, as compared with those of foreign countries, is certainly largely responsible for the fecundity of the American inventor). There are numerous instances of the spending of years of time and thousands of dollars in reducing to practical, usable form a conception of the greatest benefit to mankind, where the inventor's only sure reward depended on a monopoly of the perfected invention.

A patent is not a hindrance to the progress of any art. It gives something additional to what was known before. It takes away nothing. The art may continue to progress along its old lines if the patentee should exercise his restrictive powers to the limit. It is especially true of civil engineering works that they may generally be built in more than one way. This is a corollary of the fact that two engineers will seldom arrive independently at the same solution of a problem involving new conditions. Therefore, a patent for a new method of building a railway tunnel under the Hudson River, for example, will not give the patentee any influence in the matter, except in so far as his plan is clearly and greatly superior to others.

Engineers are often heard to decry the common practice of state and national authorities in calling for original plans with the bids for great works, and in sometimes letting the work to one bidder to be carried out according to the plan of another. This is nothing short of piracy; the appropriation of the intellectual property of another. This abuse is more often varied by the appropriation of the best features of two or more plans. Large corporations are sometimes guilty of the same offense, but the governmental authorities are the principal sinners in this direction. It is not always possible to guard against such misappropriation by the taking of patents, but the designer is nevertheless entitled to a reward commensurate with the value of the thing given, and the American Society of Civil Engineers could well establish and maintain among its members a method of procedure which would ensure a suitable reward in case of such an appropriation of the product of the engineer's intellect. The remedy by patent reaches a limited number of such cases; the others can be reached only by concerted action among the best engineers. This proposition of concerted action is not new; but, under existing laws, there is no other

Mr. Usina. remedy for the abuse, and it would not seem to be difficult to put it into effect.

It is in line with this obviously correct view of an engineer's intellectual property that he should have the right to control its use, and, in so far as patents are adapted to this end, they are a reasonable benefit. Engineers of the highest standing have taken this view. Witness the caisson and tunneling patents of Flad, Eads, Sooysmith, and Greathead; the sewerage patents of Waring; the bridge patents of Fink, Pratt, Burr, Howe, and Whipple. Some of these last show constructions which are enormities in the light of our present knowledge; but, without the work of these men and the stimulus which the patent laws gave to them, bridge building might still be far behind its present degree of perfection.

Mr. Maignen. J. P. A. MAIGNEN, ASSOC. AM. SOC. C. E.—This question is exceedingly interesting. There are three classes of engineers: the consulting engineer, the salaried engineer and the independent engineer.

The consulting engineer, like the physician, stands on a high plane, and his reputation is his patent. He is bound to give to the profession the benefit of his discoveries. His intellectual property seems to be protected sufficiently by his reputation.

The salaried engineer, who is paid to do a certain work, may, of course, have new ideas as the result of his experience. If his invention is strictly in line with his work, he, of course, owes the invention to his employer. If it is but slightly related to his work, it is still his duty to offer the benefit of the invention to his employer on terms of mutual agreement. If the employer refuses to have anything to do with the invention, or to join in the expense for its development, the salaried engineer is free to act as he thinks best, but it is clear that it is not proper for him to take advantage of the time and facilities provided by the employer in order to further exclusively his individual interests.

The position of the independent engineer is altogether different. He spends his own time and his own money (or that of his friends) in the field of research. He is not employed as a consulting engineer at $100 per day, nor as a salaried engineer at $10 000 or $20 000 per year. He works in the hope of finding a useful improvement and of interesting somebody in his invention. Perhaps he succeeds in both. A patent is taken, and he wishes to place his invention on the market. What happens then? The consulting engineer resolutely sets himself against all proprietary inventions, and the salaried engineer dares not recommend them for fear of being accused of participation in the profits.

A short time ago the speaker received a letter from an official of the Water Department of Albany, N. Y., in which he was in-

formed that the charter of that city did not allow any proposition for patented inventions to be entertained. Is it, then, a crime to take out patents? Patentees are considered as dreamers, or extortioners by some engineers. Is this right? Mr. Maignen.

In the speaker's opinion, patents are only labels for the protection of original ideas. If the patented product were not better or cheaper than the unpatented one, who would even think of adopting the former?

At the International Engineering Congress, at St. Louis, in 1904, a late lamented member of the Society gave expression to the small esteem in which patents are held by engineers, saying, in effect: "Of course, there is nothing perfect (in sewage disposal) except patent processes!"

Another member, to whom the foregoing was related, told the speaker of a friend of his, who called on the general manager of a great railroad corporation, and said, "I have come to see you about a patent." "I have no time for patents," said the manager. The visitor retorted, "Ah! this is how you manage the affairs of the railroad; you do not want to look at anything which is less than seventeen years old. Good-by." The manager's pride was touched, the effect was produced, he called the visitor back and had plenty of time to listen to his proposition.

Among certain engineers there is a feeling which has found expression in books, that, as long as any process is patented, it should not be considered. This is wrong. Every proposition should be considered on its own merits, whether patented or not.

The independent engineer usually has not the means to exploit his inventions; he must appeal to capital. Capital cannot be enlisted unless the invention is protected by a patent, but when the money and the patent are obtained, then the real difficulty begins. All the odds are against the inventor. He is looked upon with scorn or pity. Is this as it should be? The Government takes the money of the inventor in order to protect his intellectual property for a time sufficient to enable him to recoup himself for expended resources and to make an honorable living. When the limit of time has been reached, every one benefits by the process. Where would be the great industries, such as steel, the telegraph and the telephone, if they had not been protected by patents?

Our forefathers saw the difficulty. The Franklin Institute, of Philadelphia, and La Société d'Encouragement pour l'Industrie Nationale, of Paris, have established special committees to examine and report upon new inventions, and have provided funds to reward the inventor with money premiums. In 1895 the speaker received a money prize of 500 francs, from the Société d'Encouragement, for his work in purifying water.

Mr. Maignen. Now, would it not be well for the National Government, for some State institution, or for some of the millionaires who have done so much for education, to establish such a society in the United States, to reward inventors in the beginning, before they are disheartened by the coldness and opposition of interested parties? Their requirements would be small. The independent engineer, or inventor, does not want millions; he would be very well satisfied with what is now given for a short time to consulting or salaried engineers. The inventions would then be put on the market at once, with every chance of success, on account of the unbiased report and recommendation of the scientific commission which had ascertained their value, and the whole world would benefit by them without delay.

Mr. Usina. D. A. Usina, Assoc. Am. Soc. C. E. (by letter).—Mr. Maignen takes the only logical view of the rights and duties of the salaried engineer and the independent engineer. The prejudice against patented inventions, of which he speaks, is more widespread than persons of intelligence and good sense would suppose. The writer recently received a request to join with other patent attorneys in opposing the passage of laws, introduced into the Legislatures of the enlightened States of New Jersey and Ohio, providing that no patented invention should be purchased by the States under any circumstances.

Concerning the consulting engineer, however, Mr. Maignen's position does not seem to be logical. His intellectual property is not at all protected; that is to say, its ownership is not at all guaranteed to him by his reputation. There is no reason why the State should withhold from him the usual reward. The public is in no degree a contributor, either in money or in work, to the creation of the invention. The only persons having any claim are the employer who furnishes the funds, and the consulting engineer who furnishes the brains.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1029.

SEWAGE DISPOSAL.

An Informal Discussion at the Annual Convention, June 28th, 1906.

SUBJECT FOR DISCUSSION:
"The Advance in Sewage Disposal."

BY MESSRS. G. C. WHIPPLE, E. KUICHLING, GEORGE W. RAFTER, W. S. JOHNSON, J. P. A. MAIGNEN, CLYDE POTTS, R. WINTHROP PRATT, JOHN W. ALVORD, JOHN H. GREGORY, H. W. CLARK, C-E. A. WINSLOW AND G. C. WHIPPLE.

Mr. Whipple.

G. C. WHIPPLE, ASSOC. M. AM. SOC. C. E.—During the last few years the advance in sewage disposal has been conspicuous and most gratifying. It has been due not so much to the introduction of new methods of treatment as to a thorough trying-out of methods already known. Theories recently advanced have been put to practical test, and the published reports of successes and failures have helped much to establish in the minds of engineers clearer conceptions of the general principles involved. Through the mist of confused notions about septic tanks, contact beds and trickling filters, primary systems and secondary beds, and through the haze of bacteriological mysteries surrounding aerobic and anaerobic processes, experience and common sense are forcing their way and developing rational and practical methods of treatment. Of the new ideas, some have been already cast aside as unsatisfactory, impractical, or too expensive, while others seem likely to sink into innocuous desuetude before many years. A few, however, have proved of such substantial benefit that they justify all the ex-

Mr. Whipple. penditure of time and money which has been devoted to the study of the subject by many investigators working in many lands.

England has taken the lead in these matters, and it is there that one must go to find the latest developments in the art. If one asks the reason for this, a fitting reply is found in the old adage: "Necessity is the mother of invention." The English rivers are small, the English cites are large and numerous, and the amount of manufacturing carried on in them is very great; consequently the streams are badly polluted, and the need of sewage purification works is pressing. Furthermore, the soil of the country is not well suited for land treatment of sewage, and in consequence English engineers have been forced to adopt other methods. This has stimulated their ingenuity, and given rise to many modifications of chemical and bacterial processes. In England, therefore, one can see more and learn more of sewage disposal at the present time than anywhere else in the world.

French and German sanitarians have not been idle, however, and the experimental work now being carried on is sure to be followed by the construction of important works suited to their local needs.

In America the theory of sewage purification processes has been assiduously studied at a number of testing stations: notably at the Lawrence Experiment Station of the Massachusetts State Board of Health; at Columbus, Ohio; and at the Massachusetts Institute of Technology. For the most part, the results of these studies agree with those made in England at Manchester, Birmingham, Leeds, Bradford, and elsewhere. Few important sewage disposal works have been put in operation in the United States during recent years; in fact, except at Worcester, Mass., and at Providence, R. I., there are no sewage purification works of the first rank, but a large plant is under construction at Columbus, Ohio, and projects are being considered for Baltimore, Md., Paterson, N. J., and elsewhere, while many small plants have been built and are in operation.

One of the signs of advancement in the art of sewage disposal is the general recognition of the fact that an economical method of purification must proceed by distinct stages. This has resulted in the division of processes into certain groups, the examination of which will add much to a clear insight of the subject.

Classification of Methods of Sewage Disposal.

The processes involved in sewage purification may be divided into three principal groups:

1.—Preparatory Processes,
2.—Purification Processes,
3.—Finishing Processes.

Mr. Whipple.

In one sense these are all purification processes, as they all tend to make the liquid sewage less offensive. Each group has many subdivisions, and the various methods are used in all sorts of combinations. The variations are chiefly for adapting the treatment to local conditions, but too often they are due to the idiosyncrasies of the designing engineers or to the pernicious activities of companies owning certain patent rights.

Classification of Methods.

1.	2.	3.
Preparatory Processes.	*Purification Processes.*	*Finishing Processes.*
Screens,	Sub-surface irrigation,	For Clarification:
Roughing filters,	Broad irrigation,	Sedimentation,
Detritus tanks,	Intermittent filtration,	Coarse filtration.
Plain settling tanks,	Contact beds,	For Bacterial Improvement:
Chemical precipitation tanks,	Sprinkling filters.	Land treatment,
Septic tanks.		Sand filtration,
		Mechanical filtration,
		Disinfection.

In sewage purification there is a gradual separation of the solid matter from the liquid; the sludge is ultimately disposed of in one way and the more or less purified liquid sewage in another. Thus two more general groups must be provided, for convenience of discussion, namely:

4.—Disposal of Sludge,
5.—Disposal of Purified Effluent.

To these may be added a sixth, namely:

6.—Disposal of Storm Water.

It is not the speaker's intention to describe each of these processes in detail, but rather to speak of certain observations, in connection with some of them, made during a recent visit to England and the Continent.

Preparatory Processes.

Raw sewage cannot be economically purified by the direct application of any single process, on account of the complex character of its constituents. A liquid which contains substances both organic and inorganic, in which a part of each is in solution and another part in suspension, and in which the suspended particles vary in size from banana skins down to the ultra-microscopic, a

Mr. Whipple. liquid which is continually altering in composition, and in decomposition as well, and is continually varying, not only in quality, but in quantity, cannot be easily rendered free from its obnoxious ingredients. Some form of preliminary treatment, therefore, is always used. This may be a mere screening or filtering out of the coarse suspended matter, or it may be a settling out of the heavier particles, in sedimentation tanks, through the influence of gravity. Chemical or even biological agencies may be made to assist the action, but, essentially, all the preparatory processes are of a physical character. They have for their object the removal of suspended matter in order that the remaining liquid may be more easily purified by subsequent treatment.

Screening.—Little need be said about screening, except that in large plants the use of revolving inclined screens with automatic cleaning devices has rendered this initial step in the process more effective than the old-fashioned bar-screens, or cage-screens, which were lifted and cleaned by hand. At Birmingham there is an inclined revolving screen which is operated by an undershot water-wheel placed in the channel conveying the sewage. It works under a 6-in. head, the flow being about 3 000 000 gal. a day. The accumulations on this screen, which has a ¼-in. mesh, are dropped into a gutter, where an endless screw pushes them to one end and deposits them in a cart. Engineers appear to be taking a new interest in the matter of screening; finer and more efficient screens are being advocated, and in the near future important developments may be looked for. In many places the screened material is burnt under boilers, mixed with coal, or in some cases with the city garbage, and the power generated is used in various ways around the works.

Roughing Filters.—Roughing filters are used but little, although they have been studied experimentally in a number of places. As a means of removing suspended matter they are far less economical and much more troublesome than settling tanks. Where they have been used it has generally been in connection with other forms of preparatory treatment. At Salford, for instance, where, in the opinion of the engineer, they form an essential part of the preparatory process, they are placed between the chemical precipitation tanks and the sprinkling filters.

Detritus Tanks.—The use of detritus tanks, grit chambers, catch-pits, roughing tanks, or settling tanks at high velocities, as they are variously called, is very common. Their usefulness depends largely upon the character of the raw sewage. When the quantity of suspended mineral matter is large, as it often is in cities sewered on the combined system, or in places where the quantities of manufacturing waste are large, detritus tanks, arranged so that they may be easily cleaned without interrupting the flow of sewage, are

advantageous. They are also helpful in small plants where septic tanks are used. Under certain conditions, where the screening process is efficient and sedimentation takes places in a manner to be described, the detritus tank may be dispensed with. In England much thought appears to have been given to the design of these detritus tanks. In Glasgow, the sewage, after passing the screens, flows through a long catch-pit of semicircular cross-section. The sludge is removed from this at short intervals by an electrically-driven bucket-dredge. This moves over the pit on a track, lifts the sludge and conveys it to a car on a parallel track beside the dredge. All this is done, of course, without interrupting the flow of sewage. Mr. Whipple.

The time allowed for settlement in detritus tanks varies from a few minutes to several hours. The sewage often reaches the works in a septic condition, and, were it not for frequent cleaning, the sludge would undergo septic action which would tend to interfere with the usefulness of the tank for sedimentation at high velocities.

Plain Sedimentation.—Plain sedimentation at low velocities, unaccompanied by the use of chemicals or by septic action, is but little used in England, although in many, if not in most, settling tanks, the chemical and biological changes are of secondary importance. In any of the preparatory tank processes the settling of the suspended matter by gravity is the controlling function. Within the last few years there has been a reaction against the too exalted ideas of bacterial agencies, as originally set forth, and a recognition of the fact that the separation of sludge from sewage is essentially a physical problem. Plain sedimentation, modified as mentioned later, is likely to play an important part in the future.

Chemical Precipitation.—Twenty-five years ago chemical precipitation was very largely used in England. Lime, or lime and iron, or alum, were considered necessary accompaniments of the settling process. In many cases this chemical precipitation was the only means of purification used, the tank effluent being discharged directly into river or sea, or turned upon land. This method, to a very great extent, has been superseded by other modes of treatment, but even to-day there are some large cities near the coast which adhere to the old system, notably London, Glasgow, and Dublin, the last-named city having new works just about to be put in operation. London has nearly outgrown her works at Barking and Crossness, and although the efficiency of the process is said to be satisfactory, yet the volume of sewage has become so large and the quantity of organic matter discharged in the tank effluent is so great that the water of the Thames becomes almost exhausted of its oxygen during the summer when the stream flow

Mr. Whipple. diminishes. This is a condition which the authorities recognize will give rise to a nuisance if long continued. In the near future there will be needed some process which will yield a purified effluent better than can be obtained by the use of chemical precipitation alone. On account of the magnitude of her sewage problem, London has taken a conservative course, and has preferred to adhere to the present method, rather than embark upon some scheme which has not been tested by experience. This seems to have been a wise policy. The early experiments at Barking, made by Dibdin and his successors, showed the efficiency of contact beds as a means of purification, and have had far-reaching effects on the general problem; yet, in view of later studies and experience, it is questionable whether the adoption of this method for treating all the London sewage would have been the most economical one. The Thames Conservancy Board and the London County Council, in view of what has already been done, can probably be relied upon to see that the river does not become offensive. Nothing is more surprising to an American engineer, familiar with the Chicago, Passaic, or Blackstone Rivers, than to find that the Thames, flowing through the heart of the largest city of the world, is clean—no offensive odors, no unsightly scum, no litter on the shores. It is an object lesson in the art of inspecting which may well be studied by American officials.

At Glasgow the condition of the Clyde is rather worse than that of the Thames at London. This, in a measure, is due to the fact that not all the sewage of the city is purified, but it has already become clear that chemical precipitation alone will prove unequal to the task of keeping the waters of the Clyde permanently clean, and it is somewhat surprising that the new works now under construction should be practically a duplication of the old plant. The present plants appear to be well operated, however, and to be doing all that can reasonably be expected of them.

The use of chemical precipitation as a preparatory process, however, is quite another matter, and instead of going out of use it seems to be gaining favor, especially in those cities where the quantity of manufacturing waste is large. In order that contact beds or sprinkling filters may operate satisfactorily and economically, the quantity of suspended matter must be reduced to a minimum, fats must be kept down, and the colloidal substances precipitated to as great an extent as possible. In some cases this appears to be accomplished more easily by the use of chemicals than by plain or septic sedimentation. The sewage of Bradford may be taken as an illustration of this, although it is an exceptional case.

Bradford is a great manufacturing city, the center of the

woolen industry. All the trade wastes, after settling, go into the sewers. The sewage is probably the strongest and most difficult to treat of any in England. The total quantity of solid matter is about 1 800 parts per million, of which 1 100 parts are in suspension. The quantity of fats is about 650 parts per million. Bradford's present chemical precipitation works are completely outgrown, and extensive studies have been made during the last few years to determine the best mode of treatment. These studies have shown that the sewage cannot be prepared for treatment on contact beds or on sprinkling filters by either plain or septic sedimentation. Even with the septic period prolonged to two or three days the results were unsatisfactory. The use of chemicals in such a case seems to be absolutely necessary. The chief difficulty in treating the sewage is due to the excessive quantity of fats present. For this reason sulphuric acid has given better results than lime or other chemicals. The acid decomposes the soaps and separates the fats, part of which rise to the surface, where they are skimmed off. The remainder settle with the sludge, from which they are ultimately recovered by steam pressing and distillation. This grease recovery process is more extensively carried on at Bradford than anywhere else in England, and is said to be practically self-supporting. Some sludge, instead of being used for grease recovery, is mixed with coal and burnt under the boilers. The plans for the new works are not fully formulated, but it is understood that in general the process will consist of chemical precipitation followed by sprinkling filters. Mr. Whipple

At Rochdale, also, chemical precipitation is used as a preparatory treatment. Here, also, the sewage contains wool waste, and sulphuric acid is used during the day. Sulphate of alumina is also used. At Hyde septic tanks are now followed by sprinkling filters, the use of chemicals having been given up. A return to the use of chemicals during the day is being contemplated, however, as a part of the preparatory process.

The use of "alumino ferric" or "ferrozone," both of which are merely crude forms of sulphate of alumina, is exceedingly common in small plants. Thus, within the jurisdiction of the Mersey-Irwell Joint Committee, there are 70 out of 98 works at which alum, or some other chemical, is used in connection with preparatory sedimentation processes.

Although in few instances have septic tanks been given up for, or been supplemented by, the use of chemicals, engineers and inspectors appear to be coming back to the idea of using precipitants in connection with the preparatory treatment of sewages heavily laden with manufacturing wastes.

Septic Sedimentation.—The septic tank is still the subject of

Mr. Whipple. more or less controversy in England. It still has its enthusiastic advocates and opponents, but present-day discussions are tempered with reason, and are based more on fact and less on theory than in earlier days. One thing seems to be certain, the septic tank has earned its right to exist. It has not accomplished all that its originator first claimed; it does not "do away with sludge"; it does not fit the liquid sewage for further purification; it is not without nuisance. But, on the other hand, if it does not entirely eliminate sludge, it reduces its quantity, and alters its character; while, if it does not help future treatment of the liquid sewage, it at least does not hinder it. Its chief advantage is its effect on the sludge; a disadvantage is the frequent nuisance from odor.

That the septic tank is a nuisance in many places, appears to be well established. On this point the opinions of the inspectors and of residents near the works are more to be trusted than those of the sewage-works managers, although even they often complain of bad odors. The extent of the nuisance depends altogether upon local conditions—the character of the sewage, the size of the plant, the period of septicity, the nature of the subsequent purification process, the topography of the surroundings, the weather conditions, etc. It would be unjust to suppose that the septic tank is always and necessarily a nuisance to neighboring residents. Often it is not.

The septic process is an anaerobic or putrefactive one, and putrefaction, of course, involves the formation of odoriferous gases and offensive substances. Whether or not these become a nuisance depends upon their amount and the opportunity which is given them to escape into the atmosphere. Covers for septic tanks, which were part of the original idea, prevent, to some extent, the spread of bad odors, but covers are not necessary to the efficiency of the process, are expensive, and even dangerous on account of the explosive character of some of the gases liberated. Already several fatal accidents have occurred in England. Open septic tanks are often free of nuisance. In winter the septic action is not as vigorous as in summer, and the products of decomposition are less, while in summer the septic tanks are sometimes covered with a firm dense scum, which to a certain extent prevents the gases from diffusing into the atmosphere.

As an illustration of the density of the scum, the speaker might mention the case of a small septic tank which he saw at Birmingham, in which grass was growing on top of the scum. While commenting on this and on the entire absence of odor, the manager's dog strolled leisurely out on the tank and crossed to the other side. In fact, it is said that at Leeds men have crossed over the septic tanks on the scum.

When the odors do not escape from the tanks at the surface, the liquid effluent is likely to be offensive, and if this effluent is sprayed into the air or poured in thin films over a sprinkling filter, full opportunity is given for the liberation of bad odors. In fact, it may be said that the most serious nuisance arising from the septic tank is when it is used in connection with a sprinkling filter. Sanitary inspectors and representatives of the Local Government Board and the Royal Commission on Sewage Disposal were all emphatic in expressing their ideas as to the amount and frequency of bad odors due to this combination. From conversation with these officials, and from personal observation, the speaker gathered the opinion that under English conditions sprinkling filters treating septic sewage cause enough nuisance to depreciate substantially the value of property for residential purposes within a radius of one-quarter of a mile from the works, in the case of small plants, or almost half a mile from the works, in the case of large plants. If the sewage works are situated in a valley down which the prevailing winds blow in summer, the nuisance occasionally extends to greater distances in certain directions. Every plant must be judged for itself, however, and the foregoing estimate has only a very general application. In America, with more dilute sewages, the circles of influence might be less extensive. Meteorological conditions have a very important effect on the transmission of bad odors. On warm, muggy days, with a moderate breeze, a sewage plant may be a decided nuisance, while during clear, cool weather it may attract no attention whatever. This fact is too often overlooked by visitors to sewage works. Mr. Whipple.

Trade wastes sometimes tend to increase the disagreeable odors of septic sewage. This is particularly true of brewery and distillery wastes, which contain large quantities of organic matter, sulphur compounds, starch, etc. The period of septicity is perhaps even more important. Much is heard of late about oversepticity, and, unquestionably, with the strong English sewages, the 24-hr. period required by the Local Government Board is often too long. With the weaker sewages in America, and with septic periods properly adjusted, the bad odors due to the spraying of septic sewage may be somewhat mitigated.

Practical experience with the septic tank has shown that it tends to reduce the quantity of sludge. The suspended mineral matter in sewage, of course, is unchanged in the septic tank, although much of it settles through the action of gravity. A large part of the organic matter is also unaffected by the bacterial process, or, at any rate, remains in the tank as organic matter, though perhaps modified in character. Some of the organic matter in suspension, however, is liquefied or gasified, or, to use a common term, is digested. The quantity of digested matter may be taken ordi-

Mr. Whipple. narily as between one-quarter and one-third of the suspended solids, and therefore the quantity of sludge is lessened by that amount. Some engineers believe that a 15% saving of sludge is all that can be expected of a septic tank through digestion. Much, of course, depends upon the character of the sewage. Apparently, domestic sewage is better adapted to the septic treatment than sewages rich in trade wastes.

Long standing in the septic tank tends to make the sludge more compact. Speaking roughly, it may be said to reduce the volume from one-third to one-half. The character of the sludge becomes altered. This was strikingly illustrated to the speaker by Mr. John D. Watson, Engineer of the Birmingham Sewage Works. With a long pole he withdrew portions of sludge from a settling tank which had been in operation only one week and from a septic tank which had not been cleaned for three years. The former was fibrous, gray in color, and had a marked offensive sewage odor, while the latter was black, granular and almost odorless. Mr. Watson stated that the septic sludge could be spread upon land without offense, while the application of the sludge from the plain settling tank gave rise to very disagreeable odors. To do away with the latter, he has been for some time in the habit of pumping his settled sludge into septic tanks for further treatment.

Granting that the liquid septic effluent has no virtue, and may be a nuisance when applied to sprinkling filters, and granting that the use of the septic tank has a beneficial effect on the sludge, a logical improvement would appear to be the modification of the manner of using the septic tank so as to maintain the advantage and eliminate the nuisance. This is a matter now being considered by some of the English investigators. The best method of accomplishing it appears to be by the use of plain sedimentation for the sewage and septic treatment for the sludge alone. In other words, the idea is to treat the liquid portions aerobically and the solid portions anaerobically. This is scientifically correct in principle, as the beneficial anaerobic action is concerned with the carbohydrates, which are chiefly in suspension. In no case with which the speaker is familiar has this result been actually accomplished, but in a number of cases the idea has been partially applied. In the case of Birmingham, just mentioned, the sewage passes somewhat rapidly through settling basins which are cleaned at frequent intervals, most of the liquid sewage being allowed to flow to certain septic tanks, while the sludge is pumped to other septic tanks for further digestion. This takes care of the sludge, but it does not leave the sewage unseptic when it goes to the sprinkling filters.

At Hampton, England, Ault and Travis have devised what is called a hydrolytic tank. This is a tank with upper and lower

compartments. The raw sewage passes laterally through the upper compartment, while the sludge, together with a small percentage of the liquid, settles into the lower compartment, where it goes through the septic process. But in this plant, also, the liquid sewage becomes septic before it is applied to contact beds. In Macclesfield and elsewhere there are tanks for the storage of sludge, where septic action goes on. There seems to be no reason why the separation of the liquid sewage from the sludge may not be made in such a way as to leave the former in a stale, or non-septic condition, so that it may be applied to sprinkling filters with the least possible nuisance. Mr. Whipple.

Results of Preparatory Processes.—The object of the preparatory process, therefore, is to remove as much of the suspended matter as possible. It cannot be expected to remove it all. The quantity left in the sewage will depend upon the quantity in the raw sewage and the character of the process used.

Generally speaking, the preparatory treatment should reduce the quantity of suspended matter to at least 100 parts per million, and preferably to a lower figure. In the case of some of the heavy English sewages, this is not always accomplished, even with the aid of chemicals. The more sludge that can be taken out by the preparatory processes without resorting to the use of chemicals, the cheaper will be the cost of the entire treatment. When it is necessary to use chemicals, however, it becomes a nice calculation to strike the most economical balance of parts.

After the preparatory processes, the treatment of sewage is conducted along two lines which must be followed separately, that is, the purification of the liquid sewage containing the residual suspended matter, and the disposal of the sludge.

Purification Processes.

All processes which essentially alter the nature of sewage are bio-chemical. Bacterial methods of treatment are not new. The changes which took place in sewage by the older methods of subsurface irrigation, broad irrigation, and intermittent filtration were just as much bacterial as the newer methods of treatment by contact beds and sprinkling filters. The only difference appears to be that in the older methods the bacterial processes were not watched, while in the latter they are carefully controlled.

Land Treatment, Etc.—There is no need at this time of rehearsing the story of the purification of sewage by land treatment, as all matters regarding irrigation and intermittent filtration are familiar to American engineers. The new ideas relate to contact beds and sprinkling filters, both of which are essentially aerobic*

* In contact beds there may be at times a slight anaerobic action.

Mr. Whipple. processes carried on in comparatively coarse-grained beds, but which differ materially in their manner of operation and in the disposition of the sludge.

Lest the omission of intermittent sand filtration be misconstrued, it may be remarked that, in the opinion of the speaker, this is the most satisfactory and economical method of disposal where local conditions permit it to be used.

Contact Beds.—A contact bed consists of a water-tight compartment, generally built of concrete, filled with comparatively coarse material, such as coke, clinker or broken stone, and having, of course, relatively large spaces between the grains. Sewage is applied to it at regular intervals of a few hours until the bed is full, a brief period of rest is given, after which the outlet gate is opened and the liquid allowed to drain away slowly. Then the bed stands empty for a certain period, and the cycle begins again. Contact beds are often operated in series, the effluent from the first being allowed to flow to a second, and the effluent from the second to a third. The secondary beds are made of finer material than the primary beds, and the tertiary beds are still finer. The sewage is generally applied at the top, but in some instances at the bottom. Surface distribution is made by wooden troughs placed in or over the material, or else the material itself is ridged into channels, or "grips," as they are called in England.

The action of a contact bed is partly physical and partly biochemical. If the process is properly carried out, the effluent from the primary beds is fairly clear, and the effluent from the secondary beds almost entirely so. This means that the suspended matter has been left in the pores of the bed. The effluent is also non-putrescent and to some extent nitrified, indicating that bacterial action has taken place. Contact beds have a controlled respiration, air being drawn into the bed as the sewage leaves it. The oxygen thus admitted acts on the suspended organic matter left in the interstices, and, under the influence of aerobic bacteria, changes it, or a part of it at least, to the condition of a stable humus, with the formation of nitrites and nitrates. When the beds stand full the oxygen may become exhausted, and for a brief interval anaerobic conditions may prevail. It is upon the oxidation processes, however, that the actions of contact beds chiefly depend.

The largest contact beds in existence are those at Manchester, England, where they cover an area of 46 acres. They operate at the rate of about 450 000 gal. per acre daily. There is only one series of beds in use, but it is intended to build secondary beds at an early date. Septic sedimentation is used for the preparatory treatment. The contact method has been in use since 1901. The effluent is discharged into the Manchester Ship Canal. Its quality

has not been up to the standard of the Mersey-Irwell Joint Committee, that is, 1.0 gr. per gal. of oxygen consumed in 4 hours, although the degree of purification has been fairly satisfactory for single contact beds. The effluent from the one secondary bed now operating is good. Mr. Whipple.

At Burnley there are both primary and secondary contact beds, and although the plant is not as large as that at Manchester, it is in some respects a more interesting plant to visit. Here the effluent from the secondary beds is generally within the requirements of the Mersey-Irwell Joint Committee. The plant has been in operation for about eight years.

The rate of filtration of a contact bed depends upon the character of the sewage, the size of the grain, the depth of the bed, the amount of clogging, etc. A common rate in England is 500 000 gal. per acre daily; with weak sewages and with efficient preparatory treatment it may be nearly double this quantity, but with the strong English sewages it is sometimes unsafe to apply more than 300 000 gal. per acre per day. The rules of the Local Government Board prescribe that the rate shall not exceed 325 000 gal. per acre daily for each foot in depth when the effluent is to have subsequent treatment. Double contact beds are required when it is not. Contact beds naturally lose their capacity as they become clogged. After a time it becomes necessary to remove the filtering material, wash off its accumulations, and restore it to the beds. The coke or clinker filling is generally cleaned by washing and screening through cylindrical revolving screens, which separate the material into different sizes. A washing apparatus with suitable conveyors is just as essential a part of a sewage purification works where contact beds are used as is a sand washer in a modern filtration plant. Thus far, those which have been used have been crude and of a temporary character, and there is a chance for decided improvement in their design. The frequency with which contact beds have to be renovated cannot now be determined with certainty, as many factors are involved and the data are as yet insufficient. There seems to be a feeling among English engineers, however, that primary beds operating at average rates on ordinary domestic sewage may be counted upon to last from 3 to 7 years, while secondary beds may last from 8 to 15 years. At Manchester no renewals were made until the beds had been in operation nearly 5 years, and even now they are renewing only one-third of each bed, namely, those parts under the grips, where the clogging naturally is greatest.

Properly operated, double contact beds yield effluents which are reasonably free from suspended matter and color, and which are non-putrescible. Many contact beds have failed because they have been overworked or because the regimen of operation has not been suited to local conditions.

Mr. Whipple. Contact beds have one advantage over other purification processes. Their depth can be varied at will, as their efficiency is not as dependent upon depth as in the case of sprinkling filters. They have been built as shallow as 2 ft. When sewage works are located so that there is little available head, therefore, contact beds are especially applicable. They can be operated, also, with less nuisance from odor than sprinkling filters, especially when filled from the bottom. For small gravity installations, contact beds are useful, as they can be made to operate with comparatively little attention.

Sprinkling Filters.—Perhaps the most important advance in sewage disposal during the last few years is the general recognition of the sprinkling filter as an efficient means of purification. This is also called the "trickling filter," "continuous filter" and "intermittent continuous filter." The term "percolating bed" has also been suggested, as well as "spray filter." A sprinkling filter, like a contact bed, is composed of relatively coarse material. Its operation, however, is very different. Instead of being worked on the fill-and-draw principle, the sewage is applied continuously at the surface, either as a spray or in a thin sheet. The outlet gate is kept open, and the sewage trickles down through the bed and passes out of the under-drains, which are made large and open so as to insure a thorough aeration of the entire bed. The process is emphatically an aerobic one. Air is continuously entering the bed with the water, and passing in or out through the under-drains, and sometimes through the sides of the filter. As the liquid flows rapidly through the bed, and as no sewage is supposed to accumulate in the bed, there is really no need of side walls, and in some cases these are omitted, the material being allowed to take its natural slope. More often, however, loose walls of brick or stone are built up to retain the material.

The filtering material is generally coke, clinker, or broken stone. There is some difference of opinion as to the best size to be used. Some engineers prefer what are called "fine-grain beds" in which the diameters of the particles lie between $\frac{1}{8}$ in. and 1 in. Thus, at Hanley, which is perhaps the best illustration of a fine-grain sprinkling filter, the material consists of three equal layers which together make a depth of 5 ft. The coarsest, which is at the bottom, is screened between $\frac{3}{4}$ and 1 in., the middle layer between $\frac{1}{4}$ and $\frac{1}{2}$ in., and the top layer between $\frac{1}{8}$ and $\frac{1}{4}$ in. The material used is known as "saggers," and is obtained by crushing the broken pottery jars in which china is baked. More often "coarse-grain filters" are used, and material screened between 1 and 3-in. screens is very common. Both fine-grain and coarse-grain filters have their advocates, but the latter appear to be generally preferred. Fine-grain filters yield clearer effluents, but, as a result, they become clogged,

while aeration is rendered more difficult. Coarse-grain filters, on the other hand, do not clog as readily, and therefore their life is longer and the ventilation better, but the effluent is more or less turbid. Mr. Whipple.

The application of the sewage to the bed is a matter of great importance, and one which has been carefully studied by English engineers. Many patented devices are on the market for accomplishing it. As aeration is important, the more the applied sewage can be subdivided into small drops the more efficient will be the process. One of the earliest ways of applying the sewage was to allow it to flow through small holes in troughs placed over the surface of the bed. This method has been found to have its disadvantages. The holes are likely to become filled up, and the troughs tend to get out of level, thereby causing unequal distribution of the sewage.

Rotary sprinklers are much in vogue in England. They consist of four arms mounted on a central vertical shaft so that they revolve in a horizontal plane immediately above the surface of the bed. The arms are perforated only on one side, and, the sewage being discharged through them, the apparatus is made to rotate, on the principle of Barker's mill. These rotary sprinklers are now so common in England, especially for small plants, that they have become a characteristic feature of the landscape as one looks from the car windows in certain sections of the country. Some of the largest rotary sprinklers are 120 ft. in diameter, and are operated with power instead of in the manner above indicated. In some cases the end of the sprinkler is carried on a circular track laid around the bed.

Some of the objections which have been raised against the use of rotary sprinklers are that the holes tend to become clogged, and that the wind interferes with their rotation. They also require circular instead of rectangular beds, although in some cases several rotary sprinklers have been placed in large rectangular beds, the dead corners being sprayed from fixed nozzles. The intermittent application of sewage, which is obtained by the use of the rotary sprinkler, is thought by some to have decided advantages.

Instead of using rotary sprinklers, the sewage is sometimes applied to rectangular beds through a perforated pipe mounted so as to travel back and forth laterally. This arrangement is used at Hanley. The pipe is carried at the ends on trucks which run on rails along the sides, and are operated by electricity.

The movable devices appear to be satisfactory for small installations, but cannot be applied to large beds. When filters reach a size of an acre or more, permanent jets give a better distribution of the sewage. The first large plant of this kind to be erected was

Mr. Whipple. that at Salford, England, but the beds which are being constructed at Birmingham surpass it in area. The sprinklers there used are said to give general satisfaction, but the managers say they gradually become clogged, and that it is necessary to clean them out at frequent intervals. The strainers recently devised for the Columbus plant appear to be a decided improvement over any in use in England. The sprays are generally operated under a head of about 4 ft., and the applied sewage usually rises to a height of 1 or 2 ft. above the surface of the bed.

The purification which takes place in the sprinkling filter is due entirely to the action of aerobic bacteria. The suspended matter in the applied sewage settles on the grains, as the liquid passes through, until gelatinous films are formed. Being liberally supplied with air, the organic matter gradually undergoes decomposition, and changes its character, becoming somewhat granular and more like the condition of ordinary top soil. When the accumulation has reached a certain point, films of this accumulated matter break away from the grains and are discharged with the effluent. To a certain extent this process is continually going on, so that the effluent is never wholly free from turbidity, but it has been noticed that there is an unloading of the accumulated sediment at more or less regular intervals. This may be caused by an accelerated rate, by conditions which affect the bacterial action, and by the action of worms. This unloading of a sprinkling filter, which was at first thought to be a serious objection, is now believed to be its most important advantage, as it prolongs the life of the bed. There are coarse-grain sprinkling filters now in use which have been running 7 or 8 years, and are apparently in good condition. How much longer these filters can be operated will be noted with interest by engineers. It would be a mistake to suppose that they will last indefinitely, but if the material is properly chosen and the sewage is applied at the optimum rate, there seems to be no reason why they may not run from 10 to 15 years before they are renewed.

The rate at which sewage can be applied to sprinkling filters varies according to the character of the sewage, the depth of the filter, the size of the grains, etc. Deep filters can operate at higher rates than shallow filters, and are generally to be preferred. Filters less than 5 ft. deep do not operate satisfactorily, as the time required for the sewage to flow through them is too short. On the other hand, filters deeper than about 12 ft. may involve difficulties in ventilation and in matters of engineering. Depths of from 7 to 8 ft. are generally preferred. Experiment has shown that between 5 and 12 ft., the quantity of sewage which can be applied in a unit of time, varies approximately with the depth, that is to say, a 10-ft.

filter can take twice as much sewage as a 5-ft. filter. The rate which will give the most efficient purification varies inversely as the strength of the sewage. It has been found that for any depth of a sprinkling filter there is a more or less definite quantity of organic matter which must not be exceeded in a unit of time to give the best results; in other words, the size of a sprinkling filter is governed fully as much by the number of people contributing sewage to it as by the number of gallons of sewage applied. The rules of the Local Government Board prescribe a limiting rate of 325 000 gal. daily per acre for each foot of depth when the effluent is to receive further treatment, and 162 500 gal. per acre daily when the effluent is to be discharged directly to the river. Considered on the basis of domestic sewage, this would be equivalent, respectively, to an application of 30 and 15 grams per sq. m. daily, of total nitrogen. Experiments, made at Columbus, Ohio, Lawrence, Mass., and elsewhere, appear to indicate that, in order to obtain effluents which are well nitrified and are not putrescible, the amount of nitrogen applied should not exceed 25 grams per cu. m. daily, or, in other words, that each acre of a sprinkling filter should not receive sewage from more than about 2 000 people for each foot of depth. Carbonaceous matter must be considered, however, as well as nitrogenous matter, and there is an opportunity for further study in connection with this subject. Mr. Whipple.

If sprinkling filters are operated properly they will yield an effluent which is non-putrescible and well nitrified. If coarse grains have been used it will contain more or less suspended matter, for the reasons above mentioned, but this suspended matter will be very different in character from that present in the raw sewage. The purification effected by sprinkling filters seems to be generally better than that obtained by contact beds.

The chief advantages of sprinkling filters are: the purification is good, the effluents are well nitrified and non-putrescible, the filters do not become clogged except at very long intervals, and the rate of application of the sewage is considerably greater than can be obtained by any other process, thereby reducing the area required. Certain disadvantages are: the nuisance occasionally produced by spraying septic-tank effluents, and the difficulty of applying the sewage properly during cold weather. The odor nuisance may be avoided to a practical extent, however, by reducing the degree of septicity of the sewage, that is by spraying stale sewage instead of septic sewage. Experiments at Columbus, Ohio, indicated that sprinkling filters with permanent jets can be operated successfully during cold weather, but this has not yet been tested on a large scale. The idea has been suggested that in case sprinkling filters fail during the winter months they may be arranged so as to be

Mr. Whipple. operated as contact beds during cold weather. The climate of England is mild, even during winter, and therefore this is a problem which it has not been necessary to solve in that country. In the Southern States there would be no difficulty at all in using sprinkling filters at all seasons, and even in the North there is reason to believe that ways and means will be found for operating them successfully.

Results Obtained by Purification Processes.—The success of intermittent filtration and broad irrigation in purifying sewage, where suitable conditions for their application exist, has been demonstrated by many years of experience. The practical application of contact beds and sprinkling filters covers a much shorter period, yet it has been long enough to demonstrate that these newer processes may be successfully depended upon if the plants are properly designed and carefully operated. The degree of purification obtained varies greatly under different conditions. Sprinkling filters generally give somewhat higher percentage efficiencies than contact beds.

The new processes of sewage purification are sometimes spoken of as "intensified bacterial processes." That this term is well justified, in so far as it relates to the size of the works, seems evident from the figures of Table 1, which show, in a very approximate manner, the relative sizes of the plants required by the four important methods of purification.

TABLE 1.—Approximate Data as to Sizes of Purification Plants.

	Nitrogen, in grams per square meter daily.	Rate of filtration, in gallons per acre daily.	Population per acre.	Number of acres required per 1 000 people.
Broad irrigation	1	3 000 to 15 000	300	3.3
Intermittent filtration	2.5	30 000 to 100 000	700	1.4
Contact beds, 5 ft. deep	20	300 000 to 800 000	5 000	0.2
Sprinkling filters, 10 ft. deep	75	1 000 000 to 2 500 000	20 000	0.05

It will be seen from these figures that more than twenty times as much domestic sewage per unit of area can be treated by sprinkling filters as by the method of intermittent filtration. The results with the new processes, to be sure, are not quite as satisfactory, but the effluents are stable and fit to be discharged into streams not used for purposes of water supply. The figures in Table 1, showing the approximate sizes of plants, are not to be taken as applicable to any particular case, especially when manu-

facturing wastes are included. Each local problem must be studied by itself. In some instances the number of acres required may be double the figures given; often the area may be considerably less. The rate of sprinkling filters depends largely upon the size of grain used. In some recent American designs, much higher rates than those given in Table 1 have been recommended. Mr. Whipple.

Finishing Processes.

The effluents obtained from sub-surface irrigation, broad irrigation, and intermittent filtration methods require no further treatment if the plants are properly designed and carefully operated. They are usually clear enough for all practical purposes, are non-putrescible, and reasonably satisfactory from the bacterial standpoint. This, however, is not true of effluents from contact beds or sprinkling filters. Contact effluents, as well as those from fine-grain sprinkling filters, may be clear and even non-putrescible, but, bacteriologically, they are not good enough to be turned into streams which are used for public water supply purposes, or into tidal estuaries where there is danger that oysters may be contaminated. The effluents of coarse-grain sprinkling filters are not clear, but contain large quantities of suspended matter, as already pointed out.

Clarification.—A finishing process, therefore, is necessary when coarse-grain sprinkling filters are used. As the suspended matter in these effluents is comparatively coarse and heavy, it settles readily, and may be removed by the use of shallow settling basins with a short period of sedimentation, or by rough filtration through coarse material, or both. A popular method in England at the present time is to allow the sprinkler effluent to pass through a shallow settling basin, which has a narrow strip of coarse filtering material placed along the side near the outlet. The liquid is made to pass upward through this roughing filter before it is discharged. These settling basins and filters have to be cleaned at rather frequent intervals, a fact which has to be taken into account in calculating operating expenses. The sludge, however, is practically inoffensive, and may be more easily handled than that from the settling tanks used for raw sewage. In some instances the effluent from sprinkling filters is applied to land in order to clarify it, but this has usually been done in order to take advantage of land already owned. No new works are being constructed which involve this feature.

Bacterial Improvement.—Experiments made at the London works and elsewhere have shown that, from the standpoint of contamination, the effluent from sprinkling filters or from contact beds is better than that of the raw sewage, but that, while the

Mr. Whipple. organic matter is eliminated from the sewage to such an extent that the liquid is non-putrescible, yet the intestinal bacteria, and presumably the pathogenic bacteria in case the sewage is infected, are not all eliminated by any means, though somewhat reduced in number. It is not safe, therefore, to turn the effluent from a contact bed or sprinkling filter into a stream which is to be used for drinking purposes at some point below the point of discharge, without further treatment. The danger of the contamination of oysters from sewage is now well recognized, and this will have an important bearing on the character of sewage effluents in many places.

In certain cases, therefore, supplementary processes are necessary in order to eliminate the danger of infection from sewage effluents. These may consist of disinfection, or of some one of the methods used for purifying water. Broad irrigation or intermittent filtration might be used; mechanical filtration has also been suggested. Certain disinfectants, such as copper sulphate, chloride of lime, etc., have been studied experimentally, but, as far as the speaker is informed, they have not been practically used. This subject, of supplementary treatment in order to minimize the danger from infection by sewage effluents, has not yet been as carefully studied as it seems likely to be in the near future.

Sludge Disposal.

The disposal of sludge is the most difficult matter in connection with sewage works, and here progress has not been as rapid during the last few years as in connection with the treatment of liquid sewage. Managers and engineers all admit that the sludge problem is still "the problem" of sewage disposal; nevertheless, some advance has been made of late. The change from chemical precipitation to the use of septic tanks has very materially decreased the quantity of sludge to be handled, and, moreover, it has changed the character of the sludge so that it need not now create as much nuisance as formerly.

There are three principal methods of sludge disposal:

1.—Dumping at sea,
2.—Application to land, and
3.—Pressing into cakes.

Carrying the sludge to sea and there dumping it, is the most efficient means of getting rid of it. It is the most expensive, however, and is not practicable except in a few cities favorably situated.

Application to land, if it can be carried on in the immediate vicinity of the sewage works, is the cheapest method, but it is

likely to create more or less nuisance. Mr. Watson's experience at Birmingham, already referred to, is very interesting. He found that, while ordinary sludge from settling tanks cannot be spread upon land without creating offensive odors, sludge which has passed through the process of digestion in the septic tank is inoffensive when disposed of in this way. Such sludge, in one instance, was spread upon a 5-acre field to the depth of 1.5 ft., and gradually dried and settled until it consisted of a layer of what looked like ordinary top soil about 6 in. thick. It was thought at first that crops could be grown on the dried sludge, but this did not prove to be practicable unless earth was mixed with it. Mr. Whipple.

Sludge cannot be easily pressed without the addition of lime, and septic sludge is more difficult to press than either the sludge from chemical precipitation tanks or from plain settling tanks. It requires more lime and more care in operation. Nevertheless, in some places it is done with apparent success.

The value of pressed sludge seldom equals the cost of the lime and the pressing. In some places, where the pressed cake is used for manurial purposes, the sums received from the farmers just about pay the operating expenses, while in other cases the process is carried on at slight loss.

Pressed cake appears to have just about the same manurial value as an equal weight of barnyard manure. It has been found, however, that it does not give good results as a fertilizer when applied continuously year after year to the same piece of land. That is one reason why its sale usually drops off a few years after the sewage plant has been established.

Some sewage sludges are so rich in fats that it is practicable to use them for fuel, or even to recover the grease. The recovery of the grease does not interfere with the later use of the sludge as a fertilizer, as the nitrogen is not extracted. The sludge from settling basins receiving the effluent of sprinkling filters is much less offensive than ordinary sludge. It can be applied to land with practically no nuisance.

At Birmingham, England, where the sludge appears to be well handled, for an inland city, it is said that the general level of the land in the vicinity of the settling tanks is being raised at the rate of about 1 in. a year.

Disposal of the Purified Effluent.

The object of purifying sewage is chiefly to prevent it from being a nuisance to sight or smell when discharged into some body of water. In certain instances it is further demanded that the effluent shall be satisfactory from a sanitary point of view, in order to avoid the danger of contaminating water supplies or oyster beds.

Mr. Whipple. The subject of sewage disposal by dilution and the effect of sewage effluents on the quality of the water in streams and tidal estuaries is so broad that it cannot be brought within the scope of this discussion, and it is listed merely that it shall not be lost sight of in a general discussion of the subject.

Disposal of Storm Water.

As so many English cities are sewered by the combined system, the disposal of storm water is an important part of any system of disposal. The rules of the Local Government Board of England require that sewage works shall be large enough to treat twice the dry-weather flow, in the case of separate systems, and three times the dry-weather flow, in case of combined systems, at the rates mentioned previously. A further treatment of storm water is required, up to 4 volumes of dry-weather flow, in the case of separate systems, or 3 volumes, in the case of combined systems. The prescribed minimum rate of treatment on storm beds is 2 900 000 gal. per acre daily. In both cases, therefore, storm flows must be treated in one way or another up to a volume of 6 times the dry-weather flow. Any excess above this may be discharged directly into a stream without treatment.

Storm water is very commonly treated on beds of coarse material operated continuously as long as the storms last, but having long periods of rest between. In order to keep the beds in good condition, however, they are often allowed to receive sewage in small quantities during the dry periods, but care has to be taken that they are not clogged when needed for full duty. During times of storm the period of sedimentation is much reduced, so that suspended matter is often carried to the storm beds in rather large quantities. This, together with the grease which is often present in large quantities, may give rise to surface clogging. Storm beds, therefore, require to be scraped at frequent intervals, or to have the upper layer of material removed and washed.

Instead of using coarse-grain beds, storm water is sometimes turned upon land. It is often the case, when contact beds or sprinkling filters have replaced the older process of land treatment, that the old areas are reserved for the treatment of storm water.

Mr. Kuichling. E. Kuichling, M. Am. Soc. C. E.—The excellent presentation of the subject of sewage treatment which has been given by Mr. Whipple is a valuable addition to our stock of knowledge, as it comes from one who has devoted much time to the study of the problem, and whose previous work in chemical and biological fields makes him unusually well qualified to observe and criticize the results attained in this respect in England. The scope of the discussion, however, is so extensive that his paper has left room for

some additional remarks, which are offered in the hope of eliciting opinions that may be of service to those who are engaged in planning or operating sewage disposal works. Mr. Kuichling.

In most of the papers and reports relating to the treatment of sewage, reference is made to the varying composition or character of the liquid, and to the effect of adding trade wastes to those derived from households. In general, such addition is regarded as rendering the task of purifying the liquid more difficult or expensive, especially when the percentage of trade waste is large. In growing cities the nature of the manufactures often changes, especially in the smaller towns; hence it becomes desirable to devise a plant in which the process of treatment can be varied or modified from time to time, in accordance with changes in the character of the sewage delivered.

The difficulties of treatment increase when the trade wastes contain considerable quantities of fat, soap, dyestuffs, poisonous metallic salts, free inorganic acids, refuse from abattoirs, tanneries, breweries, gas-works, textile mills, etc. On the other hand, a moderate quantity of salts of iron in solution, but containing no free acids, is often regarded as a beneficial addition to domestic sewage, since such salts act as a chemical precipitant of some of the organic matter, and tend to deodorize the liquid. In biological purification, however, it seems to be a matter of much importance that all the iron shall be precipitated in suitable tanks before the liquid is applied to contact beds or filters, otherwise the durability and efficiency of the latter is greatly impaired.

From these considerations it follows generally that the greater the quantity of trade waste, the more complicated must be the plant and the process of treatment, hence the recent tendency of subjecting the sewage to a preliminary chemical precipitation before applying it to contact beds or filters for final purification.

Furthermore, in all operations connected with sewage disposal works, the avoidance or prevention of the formation and dissemination of offensive effluvia must constantly be kept in mind. This feature is, perhaps, the most perplexing and troublesome of all the problems encountered in sewage treatment, as the development of odors from decomposing organic matter obviously varies with the temperature and humidity of the air, while their distribution depends on the wind. It is often stated that fresh sewage has very little odor. This may be true for a short time after its production, but it is a matter of common knowledge that the air contained in municipal sewers is always unpleasant and often becomes highly offensive. The fact is that by the comminution of the decomposing matter in passing rapidly through the waste pipe from the point of production in the building to the street sewer, a vastly greater

Mr. Kuichling.

emanating surface is formed, and also that the process of decomposition or putrefaction is not arrested by submerging the matter in water.

It follows, therefore, that the longer the matter remains in the sewer, the stronger will be the odor of the sewage, unless the liquid receives, on its way, an addition of trade waste which tends to destroy the offensive gases or effluvia, as in the case of Leeds, where these wastes contain a large percentage of salts of iron. For this reason, also, sewage which has to travel through a long conduit usually arrives at the disposal works in a more odorous condition than while it was in the tributary sewers. In cool weather the development of effluvium is reduced, and may become slight in winter, but it rarely happens that the odor disappears entirely; hence it becomes expedient to locate the disposal plant as far as possible from habitations. Another reason for such location is the fact that sewage works attract multitudes of flies and other insects, which sometimes become sources of serious annoyance to persons residing in the immediate vicinity.

Even in the rare cases where the sewage arrives at the plant without much offensive odor, trouble will frequently be caused by emanations from the tanks and beds in which the process of purification is carried on, especially when these are of large extent. The appreciation of an odor depends largely upon the magnitude of the emanating surface, so that the effluvium arising from an area of a few square rods may be insignificant, while that which comes from an area of 10 or 20 acres may become intolerable. In the latter case either the plant must be removed, or the adjoining lands must be bought and made part of the works, or else the tanks and beds must be covered, and the effluvia carried by chimneys far enough above the surface of the ground to avoid nuisance. This latter process is usually very expensive, and has not yet been undertaken in extensive works.

As stated by Mr. Whipple, the coarse suspended matter in sewage is always removed by mechanical means in order to reduce the quantity of sludge as much as possible. Various devices have been tried, but the simple inclined grating with bars about 1 in. apart is commonly used in the larger English plants. Few data concerning the quantity of material thus extracted are available, and the only record that the speaker has now at hand is that relating to the screens at Charlottenburg, where, in 1902, approximately 1 cu. m. of solid matter was obtained, on the average, from every 13 250 cu. m. (3 500 000 U. S. gal.) of sewage. There is also a tendency toward obtaining a higher efficiency of the detritus tanks in which sand, pebbles, cinders and other heavy mineral matter settle before the sewage reaches the large sedimentation tanks. At Manchester,

from a dry-weather flow of about 29 000 000 U. S. gal. per day, the screens and detritus tanks yield about 90 tons of matter per week; but in times of heavy rain they have yielded as much as 140 tons in a single day. Mr. Kuichling.

Most of the recent investigations of the subject in England and Germany have led to the conclusion that from two-thirds to four-fifths of the putrescible organic matter, which is carried in suspension in the sewage, can be removed by simple sedimentation, and that, in many cases, the effluent from such tanks may be safely discharged without further treatment into relatively small rivers, when the same are not used for water-supply purposes. This opinion was also entertained years ago, after some experience had been gained with sedimentation alone, but it was crowded aside by the promoters of various chemical processes, as well as by those who sought for still better results by other means. It is, therefore, interesting to note the revival of some former doctrines and the confirmation of their soundness by the newest methods of research.

Associated with the efficiency of simple sedimentation is the old project of attaining the same end mechanically by screening the sewage through a sieve having very small meshes. Many experiments in this direction were made in former times, but were abandoned because an entirely clear effluent could not be produced thereby. The demonstration of the fact that, in many cases, clearness and sterility of effluent are not essential features of sewage treatment, will probably also lead to the trial of other screening devices in the hope of avoiding the great expense of existing processes.

Where chemicals are used for precipitating the suspended matter, the quantity ranges in general from 2.8 to 9.4 gr. per U. S. gal., or from 400 to 1 340 lb. per million gallons. Lime is used at rates ranging from 2 to 5 gr. per gal.; sulphate of iron or copperas at from 0.82 to 4.67, and alum or compounds of iron and aluminum sulphate at from 1.29 to 7.01. The quantity used varies with the character and strength of the sewage in each case, and also with the nature of the trade waste; and in most cases a variation in the rate of admixture is made at different hours of each day.

No settled practice seems to have been established in regard to the form and size of the sedimentary and finishing tanks and the total storage volume to be provided. In 1903 the requirements of the Local Government Board were that the works should be capable of purifying, in a satisfactory manner, at least six times the daily dry-weather flow, and that three times this flow should be purified in the same manner as when no rainfall occurs. When the discharge is three times the dry-weather flow, the tanks of a double-treatment system may be emptied twice and filled three times in

Mr. Kuichling. the course of 24 hours, so that the storage capacity shall be equal to the daily dry-weather flow. In addition to this standard capacity, one or more storage units must be held in reserve for use in case of repairs to some of the others. Where the purification is done in only one set of tanks, the storage capacity should be equal to 1.5 times the dry-weather flow. It may be remarked, however, that these requirements are not usually fulfilled, and that, in the majority of cases, the storage capacity is not more than one-half the daily dry-weather flow.

The most important result of the recent investigations, however, is that the new methods of treatment have not succeeded in eliminating the production of sewage sludge. While it is true that an appreciable reduction of the quantity of sludge is effected by the septic process, yet enough of this objectionable matter remains to cause serious embarrassment and expense in large plants. As far as can now be foreseen, it will be necessary to remove such matter periodically from both sedimenting and septic tanks, as well as from contact beds and filters. The frequency of these operations obviously depends on the character of the sewage and the trade wastes, and the data are not yet sufficient to admit of the formulation of reliable estimates in this respect. It may be said, therefore, that the new processes have been somewhat disappointing, and that there is still room for much improvement.

Mr. Rafter. GEORGE W. RAFTER, M. AM. SOC. C. E. (by letter).—In view of the discussion before the International Engineering Congress, at St. Louis, in 1904, this topic might be more properly stated as, "Has there been any advance in sewage disposal," and does it mean advance in sewage disposal in the United States, or in England, or the whole world, or where? As far as the writer is concerned, the topic will be discussed chiefly from the point of view of advance in sewage disposal in the United States in the last few years.

Engineers must go to England for the bulk of the information about sewage disposal, although fifteen years ago, the State Board of Health of Massachusetts, with its classical experiments on intermittent filtration, for a time led in this specialty. Later, the palm went to England, and has since remained there, the most of the work done here in recent years being largely on the lines laid down by the English experimenters, so that, to-day, if one wants the present practice, he must go to the report of the Royal Commission on Sewage Disposal, where, with an unparalleled wealth of detail, nearly every phase of the subject has been considered. For the last few years, American investigators have allowed the palm to remain in England, but it is hoped that in any further work undertaken here, new lines may be worked out so as to reverse this, to some extent.

While the foregoing is somewhat sweeping, nevertheless there are individual examples of creditable advance in sewage disposal work in America, but the statement is made that, as regards the broad lines, the advance in England in the last few years has been much more rapid and on more material points than in the United States. The writer will not discuss the relative points, because they ought to be obvious to any person familiar with sewage disposal. Mr. Rafter.

WILLIAM S. JOHNSON, M. AM. SOC. C. E.—Mr. Whipple has given a remarkably clear and interesting account of the present status of the sewage disposal problem in England, but the speaker regrets that he has confined himself to the practice there, where conditions are so divergent from those which obtain in the ordinary American city that, in many respects, the problem is entirely different. He states that one must go to England to find the latest developments in the art of sewage purification. It is undoubtedly true that the latest developments are found there, but the speaker believes that in America the highest development exemplified in intermittent filtration may be found. The great interest in recent English experiments has, to some extent, obscured this system which has been developed in America to meet American conditions, and which has proved successful in so many places. Mr. Johnson.

In a recent editorial in one of the leading engineering papers the following statements were made:

"The rise of intermittent filtration has reached its culmination. It is now known that it is much more expensive than was formerly anticipated, and the character of the effluent which it furnishes is often unnecessarily good for discharge into the body of water receiving the purified sewage. One by one the intermittent filters constructed in New England, a number of years ago, are becoming clogged and inoperative, except at very low rates, and the heavy initial expense, which is usually necessary for such filters, is recognized as merely a forerunner of heavy maintenance charges."

These statements, to one familiar with the New England intermittent filtration works, are amazing. The intermittent filtration areas in Massachusetts, almost without exception, purify the sewage satisfactorily during nine or ten months of the year. The purification is practically as satisfactory in cold weather as in warm, until the accumulation of solid matter on the beds, during the time when the surfaces cannot be cleaned on account of ice, becomes so great as to cause clogging. Not one of the plants is becoming inoperative, although several have been outgrown, and the sewage discharged upon them during the spring months is more than they can purify satisfactorily.

The only serious difficulty which is experienced in the opera-

Mr. Johnson. tion of the plants is that due to the clogging of the beds in the late winter and spring, after the surfaces have been inaccessible for several months. At such times, not only are the surfaces of the beds clogged by the accumulated solid matter, but the flow of sewage is at its maximum. To purify the sewage at such times, with plants as now constructed, it is necessary to provide an area at least twice as great as that which would be required during the remainder of the year. During the time when this trouble is experienced, the flow of the streams is high, and the discharge of unpurified sewage directly into them is likely to cause little offence. In Massachusetts, however, the State Board of Health objects to the discharge of unpurified sewage from a purification plant at any time, insisting that the plant shall be of sufficient capacity to meet the greatest demands. It is this high standard adopted by the Massachusetts State Board of Health which is probably responsible for the opinion in regard to intermittent filtration given in the editorial quoted.

The satisfactory results from intermittent filters are obtained in spite of the fact that many of them are badly neglected. This, it seems to the speaker, is one of the great advantages of the system. The plants, if well designed in the beginning, give results which are much more satisfactory than those obtained by the more rapid systems, even though neglected, as they are sure to be in many cases, especially in the smaller cities and towns, while the rapid systems require constant and intelligent care.

The intermittent filtration areas of New England are purifying the sewage of from 500 to 2 000 persons per acre—a much greater quantity than was anticipated. With improvements in construction and with the adoption of suitable works for the removal of a portion of the solid matter, it is probable that the ordinary sewage of from 1 500 to 2 000 persons can be disposed of satisfactorily on each acre of filter beds.

The cost of maintenance of intermittent filtration areas depends, to a considerable extent, upon their size, but, for the larger works, it is from 10 to 15 cents per year per person contributing sewage, and, with increased knowledge in regard to the maintenance of the beds, this figure is being decreased.

Failure to realize the difference in conditions in different places has resulted in a mistaken idea as to the quantity of sewage which can be treated by the various systems of purification. The figures usually given for intermittent filtration are for fresh sewage from which none of the solid matter has been removed. The figures given for the capacity of trickling and contact filters, on the other hand, are generally those for septic sewage or at least for sewage from which a large proportion of the solid matter has been removed. In

the experiments recently made at Columbus, Ohio, it was shown that the quantity of sewage which could be purified by intermittent filtration was about one-eighth of that which could be purified by trickling filters, similar sewages being used in each case. In the few intermittent filtration works in New England where there is a thorough preliminary treatment of the sewage, quantities of sewage amounting to 250 000 gal. per day are purified by an acre of sand filter beds. Furthermore, in comparing the work which can be done by the various systems, it must be remembered that it is essential in the case of trickling filters to construct works for the further purification, or at least clarification, of the effluent, to make it at all comparable with the effluent even from an overworked and badly managed intermittent filter. Experience in the operation of the two classes of filters tends to show that larger quantities than were anticipated can be purified by intermittent filtration and smaller quantities by trickling filters. When the cost of maintenance and the cost of renewals of the filtering material are taken into consideration, it will be found that intermittent filters can frequently compete with the so-called rapid filters, even when it is necessary to convey the filtering material for these filters from a considerable distance, and the results obtained by intermittent filtration are sure to be much more satisfactory. Mr. Johnson.

In this discussion the speaker simply wants to emphasize the fact that developments in the art of sewage disposal in England do not necessarily apply to American conditions, and that the most recent development may not be the highest. Intermittent filtration is not a stage in the development of sewage purification which has been passed or which is even passing, but it represents to-day the highest development of the art of sewage purification under American conditions.

J. P. A. Maignen, Assoc. Am. Soc. C. E.—The subject of sewage disposal is quite extensive, and the speaker will simply draw attention to a few points for further consideration, presenting these points in the form of questions. Mr. Maignen.

The first question is: Is it desirable to screen the sewage that goes into the septic tank?

If the so-called septic action consists in the decomposition of dead organic matter by bacteria and in its reduction to inorganic substances, it would appear as if the more food allowed to go into the septic tank, the more strenuous and complete would be the septic action.

It is well known that all portions of what has once been a living body, vegetable or animal, which have been cut off the trunk, or deprived of life, as well as all animal excretions, commonly called organic matter, are destined, whether they be in water, in sewage,

Mr. Maignen. or in any other moist environment, to be decomposed by certain kinds of bacteria, which, unless fed by new material, die after having done their work.

Place a bottle, or other vessel, full of fresh sewage, upon a table or shelf, and examine the bacterial process. One may find on the first day, for instance, 500 000 bacteria per cu. cm., the second day, 1 000 000, and the third day, 2 000 000. The number may go on increasing for a certain number of days; then comes the time when the increase ceases, and, on the contrary, the bacteria begin to die out; so that, supposing the number to have been, say, 4 000 000 on the fourth day, it may, by the tenth day, have fallen to 2 000 000, on the fifteenth to 1 000 000, on the twentieth to 500 000, and on the thirtieth day the number may be reduced as low as 50 000.

The offensive odor, which rises from sewage in a state of decomposition, indicates the presence and work of the bacteria.

When crude sewage is emptied into rivers, the decomposition is distributed over a large area, and the quantity of gas produced in any one place is hardly noticeable. But, under these conditions, it may be easily understood that infection can be carried a long distance. By infection is meant the entrance, into the human system, of bacteria which have been living on dead, or diseased, animal or organic matter.

Water, taken in barrels by ships from rivers known to be polluted, such as the Delaware River, becomes foul after a few days at sea, but later it becomes sweet and pure. By this time, all fermentation or putrefaction has ceased. The same thing occurs in rain-water tanks, in cider-making, and in wine-making. As long as there is appropriate food, the bacteria thrive and go on increasing until, in the last act, they all die, as in Shakespeare's plays.

The question, then, may be asked: Why should not all the material that comes from the sewage enter the septic tank? Why should any part of the food which is to help the septic action be taken out by screening?

The next question is: What manurial value, if any, is there in fresh sewage, or in the sludge or scum of septic tanks?

A few weeks ago, Dr. Samuel G. Dixon, Health Commissioner of Pennsylvania, in drawing public attention to the danger of infection from sewage, laid particular stress on its fertilizing value, which is wasted under present conditions. It is in order, therefore, to ask whether it would not be possible to do with the sewage of towns what is being done with farm refuse? Does the septic action destroy all manurial value, or does it leave some, and if so, how much? And would it not be desirable to study, more thoroughly than has hitherto been done, the utilization of fresh sewage?

The speaker is aware that this is not a new question. Thus, for

instance, sulphate of ammonia is extracted from the product of odorless excavations in Paris, and fresh sewage is thrown on farm lands in many places, but it occurs to him that there are not enough data in the United States on the whole question of the manurial value of fresh sewage, or of the residue of septic tanks. Mr. Maignen.

In Paris new fields are being constantly added for the filtration of sewage by what is known as broad irrigation, but these have not kept pace with the requirements. The rivers are more polluted than ever with the overflowing raw sewage from the sewage farms. The reason is that the fatty and greasy substances do not decompose but turn the sand or soil into a kind of putty, which prevents filtration, so that the ground ceases to absorb liquids in the desired quantity.

Of course in times of rain and frost much of the raw sewage goes directly into the rivers. It is appropriate, therefore, to ask for a thorough investigation of the question of utilizing the manurial properties of sewage, both as to what has been done, and as to what ought to be done.

As a general rule, chemicals, such as iron or copper, are inimical to vegetation, but this may not be the case with lime, which has been used extensively in sewage purification. It may be that, in those countries in which the soil is poor in lime, and where lime is actually spread on the fields to enrich the soil, the use of this earthy salt, for sewage purification, would be desirable. Lime, impregnated with partly decomposed organic matter, may have acquired some additional manurial property which would pay for the trouble of utilizing it in sewage purification. Any information on this subject might prove of interest.

The speaker would like to ask Mr. Whipple as to the cost of sewage treatment in England per million gallons by the various processes which he has described.

A thought now occurs to the speaker, which perhaps might better have been mentioned earlier in the discussion, but which is of sufficient interest to be submitted for consideration.

It has been seen that sewage and water, which contain organic matter, pass through a stage of fermentation, attended, in some cases, with the production of gas; and that after a time such sewage or water becomes odorless and innocuous; in other words, that the multiplication of bacteria is not without limit in time, space, or number. Take, for instance, a gelatine plate on which a bacterial culture is made. At first mere points or dots are seen, which are called colonies; some of these points remain small, while others grow larger and attain different dimensions, varying from $\frac{1}{16}$ to $\frac{1}{4}$ in. in diameter, and then they stop. The bacteriologists might perhaps tell us why there is this limit in growth, and why a single colony does not grow so as to cover the whole plate?

Mr. Maignen. There are some liquefying bacteria which, if allowed, would decompose the whole gelatine, but there are others which remain within specific limits.

It is supposed that the product of bacterial metabolism is poisonous to its own kind, as would be the case if too many persons were confined in a closed room where the carbonic acid exhaled would poison the people so confined. The same phenomenon probably obtains in the case of sewage, and perhaps this may be accounted as one of the causes of so-called self-purification.

In conclusion, the speaker would like to ask Mr. Whipple whether, on his last visit to Paris, he took a ride in a boat in the sewers? This used to be one of the sights of Paris, as much as the Tour Eiffel. These sewers are large enough to pass the flow of sewage and also to furnish room for water-pipes, gas-pipes, etc. When the speaker was in Paris, he understood that these sewers took only the surface water, and he would like to know whether house sewage is now allowed to mix with it?

Mr. Potts. Clyde Potts, Assoc. M. Am. Soc. C. E. (by letter.)—Mr. Whipple's paper is an exceedingly interesting discussion of the sewage disposal plants studied by him while abroad, and, as such, is a valuable addition to the ever-increasing literature on sewage disposal.

While the paper treats of English methods and their application to American cities, it does not treat of American practice, or rather of what American conditions are bringing about in contradistinction to what English conditions have evolved in England.

It is apparent to all that each municipality having a live question of sewage disposal must study its own particular needs. This study involves the concentration of the sewage, the time required to reach the disposal plant, the degree of purification consistent or desired, the quantity of manufacturing wastes, the filtering material locally available, and the financial standing of the municipality. In short, it is a complete case of individual study, just as each case that comes to a physician is an individual study.

In England the symptoms are different, and the available remedies are also different. Americans are lavish users of water, and their sewage is much more highly diluted than English sewage. In speaking of disposal plants, if one could use the terms, persons per acre or square feet per person, for filter beds, and, cubic feet per person, for sedimentation tanks, the data from different communities and cities would be more comparable, as reference to water consumption per capita would be avoided. In America, filtration is the distinctly American practice, and sewage purification could well be called "the filtration of highly polluted water," or "the filtration of dilute sewage." The term "purification of public water supplies"

seems to be a misnomer. As all surface waters are more or less polluted, it is generally conceded that all need some treatment before use. In treating these waters it is true that purification is attempted, but is it ever achieved? The time is not far distant when the field of the so-called specialist in the filtration of public water supplies (polluted water), and the specialist in sewage disposal (the filtration of highly polluted water), will be one and the same. Sewage is water, and though it be highly polluted with human excreta and waste, it is water, nevertheless, and carries a certain definite quantity of organic matter per 80 gal., per 100 gal., or perhaps per 300 gal., according to what the per capita consumption of water may be. Mr. Potts.

The great number of American sewage disposal plants that have been built, and the tendency of the times seem to indicate that filtration through sand beds, natural or artificial, is the principal and ultimate step in the attempted purification of sewage. It is Nature's own method, and if the life of the sand beds can be saved and the cost of maintenance cheapened by introducing sedimentation tanks, contact beds, coarse primary filters, scrubbing filters, sprinkling filters, etc., the final and principal step will still be filtration. The only thing achieved by these preliminary steps is the reduction of the number of square feet of filters required for each person contributing organic matter to the sewage. For each individual case in sewage disposal there is an economic limit to the size of filters, as determined by the cost of the preliminary steps leading to filtration. In some localities where preliminary steps will give a non-putrescible effluent, it is not worth the added cost to build filters; but the time is not far distant when the needs of society will compel, in all cases, the highest degree of purification possible.

R. Winthrop Pratt, Esq.* (by letter).—Mansfield, Ohio, has a population of approximately 20 000, about one-half of which is tributary to the city sewers. The system consists of 25 miles of sewers, about 60% of which are on the combined plan, and 40% are used for domestic sewage only. Automatic overflows divert a portion of the sewage directly into the creek at times of storm. Mr. Pratt.

The sewage is rather dilute most of the time, due in part to leakage into the old combined sewers; and there are no manufacturing wastes to complicate its composition. The average daily quantity of sewage treated at the disposal works is about 1 000 000 gal.

The disposal plant consists of covered septic tanks and contact beds. There are four tanks, each 50 by 100 ft., with an average depth of 8 ft. This gives a total tank capacity of 1 000 000 gal., or about 24 hours' flow. An automatic floating weir secures a con-

* Chief Engineer, Ohio State Board of Health.

Mr. Pratt. stant discharge from the tanks. This causes a daily fluctuation of about 6 in. in the elevation of the surface of the liquid.

There are five contact beds. Each is filled with 5 ft. of specially prepared cinders, and has an area of ¼ acre, making a total area of 1¼ acres.

The sewage enters the septic tank through four 12-in. pipes, 10 ft. apart and 3½ ft. below the surface of the liquid. The sewage is drawn off at the lower end of each tank through 2-in. pipes discharging into a collecting channel, outside of the main wall of the tank, about 2 ft. wide and 3½ ft. deep. The sewage flows from this channel through the floating weir into an 18-in. pipe leading to the aerator, and thence to the contact beds.

Sludge Disposal.

After 4 years' operation it was found that these outlet pipes had become clogged to such an extent that the sewage could not pass from the tanks. It was decided, therefore, by the local authorities, that cleaning was desirable. The actual deposit of sludge in the tank at the time of cleaning was only 2 ft. at the upper end and 1 ft. at the lower end. The collecting channel contained a deposit 6 or 8 in. deep.

Two theories are presented to explain the clogging of the 2-in. outlet pipes: First, that the increased bacterial action at the bottom of the tank, which takes place in the spring, caused the sludge to rise and be carried into the 2-in. pipes; and second, that the accumulated suspended matter in the outside collecting channel became so great as to cover the outlets of these small pipes and clog them. If this last theory is true, then the cleaning of the septic tanks at this time was not necessary.

The removal of the sludge from one of the tanks was effected by drawing off the supernatant sewage and then opening the sludge valve and flushing the sludge directly into the creek. This flushing was directed by men who entered the tank and aided the work when necessary. It was found that the sludge was of such a pasty consistency that it would not flow readily, but needed considerable dilution.

After cleaning the first tank, the stage of the river decreased to a decided extent, and it was not thought wise to adopt this method with the remaining three tanks. Therefore, a framework with hoisting apparatus was erected in turn over each of the manholes in the other three tanks, and barrels were lowered into each and filled by five men working therein with buckets. It is said that about 250 bbl. of sludge were removed each day by this method. Each barrel, after being hoisted to a height of 10 ft. above the top of the tank, was discharged by tipping into a sluice-way which led

down a steep embankment to a small piece of low, marshy land adjoining the works. Mr. Pratt.

The total quantity of sludge removed was about 1 200 cu. yd. The cleaning of the tanks required 5 or 6 men for 3 weeks. The cost was about 50 cents per cu. yd. Based on an average daily flow of 1 000 000 gal., the accumulation in the tanks, during the 4 years of service, was about 0.8 cu. yd. of wet sludge per million gallons of sewage.

The sludge was of a deep black color, but had no appreciable odor; that is, with the wind blowing directly from the pool of sludge deposited on the low land, adjoining the works, practically no odor was carried more than 100 or 200 ft. on a warm and clear day. Sludge, which had been accidentally spilled around the top of the manholes, turned from black to a dark brown in a few days; and then, when dry, was little more offensive than ordinary garden earth. The change of color was apparently due to the oxidation of the iron sulphide contained in the sludge.

As noted from the following analysis, this is a typical septic sludge, and apparently has been so worked over by bacteria as to render it comparatively inoffensive. The analysis shows the sludge to be very similar in composition to that found in the septic tanks at the Columbus, Ohio, Testing Station after 11 months' operation.*

ANALYSIS OF SLUDGE FROM MANSFIELD SEPTIC TANK.

First Cleaning After 4 Years' Use.

Specific Gravity	1.11
Water	80.8
Solids	19.2
Volatile	3.6
Nitrogen	1.03
Fats, acid	4.65
Iron (Fe), total	0.77
FeS	0.764
Fe, not as FeS	0.28

The experience at Mansfield is of particular interest in showing that it is possible to operate a septic tank continuously for 4 years without any serious accumulation of sludge; and that the sludge can be discharged into the open air and deposited on ground near the plant without creating offensive conditions. This seems to conform more closely with European experience, as reported in the United States, than anything which has hitherto come under the writer's observation.

Acknowledgment is made to Mr. L. A. Keith, City Engineer of Mansfield, for information relating to the cost of the work.

* "Report on Sewage Purification for Columbus," by George A. Johnson, 1905, p. 131.

Mr. Alvord. JOHN W. ALVORD, M. AM. SOC. C. E.—The speaker has been very much interested in Mr. Whipple's exhaustive statement of the present status of sewage purification. In a careful examination of the history of the art, made recently, it has been somewhat illuminating to see how many times things supposed to have been recently discovered had been discovered long before, and sometimes several times before.

Looking back along many of the lines of thought commonly supposed to be original with recent minds, it will be found, often most unexpectedly, that somebody, with great manifestation of joy, has thought that thought before, and written and talked about it, often with a great deal of interest, and often with a great deal of value to his time and place.

To take an example: Perhaps not everyone of those interested in sewage disposal has known of the work of Dr. Alexander Mueller, who, about 1869 or 1870, was City Chemist of Berlin. Dr. Mueller was very much interested in sewage disposal; he made important experiments, and developed the idea that the dissolution of organic solids is initiated by small, leaven-like organisms, the character and habitat of which he studied experimentally. Dr. Mueller published an account of those experiments, and described bacterial action in language very closely resembling some of that used in recent literature on the subject.

Coming down a little further: Mr. Louis Mouras, a Frenchman, made a sort of re-discovery of Dr. Mueller's discovery, and succeeded in finding a friend who had imagination and a journal, and there may be found his ideas of bacterial ferment described in very clear language—language which resembles that used in the descriptions written in recent years by Mr. Scott Moncrieff, Professor Dibdin and Dr. Rideal, on the same subject.

Mr. Mouras had an opportunity to try his invention extensively in Paris, and with quite a good deal of success. Then Signor Paglioni, in Italy, seems to have picked up the same line of thought, and worked at it; and Col. George E. Waring, Jr., in America, seems to have been early in the field; and, even in England, Sir Henry Austin, as far back as 1852, described apparatus for the purification of sewage, and the different stages of reduction, in language which is quite readable to-day, and would appeal to intelligent men as being in line with much that has been written recently.

And so it is seen that this is really a very old subject, one that has been worked over by a great many minds, in many places, and at many times, and to-day we are joyfully re-discovering a great many things which have been somewhat well discovered before.

At the same time, there is one interesting episode, which de-

velops from a study of the history of the art, which is impressive, and that is the fact that it is possible for the thought of the time to be led away from the best lines of development by misdirection. Mr. Alvord.

The discoverers who have been mentioned developed their ideas about innate bacterial action in the infancy of the era of bacterial thought, and even prior to its general acceptance; and while it is now considered that their work was along right lines, their ideas, although published, were not always absorbed into the active discussion of the time, or, where discussed, often failed to impress their importance properly upon the majority of contemporary minds. Thus, curiously enough, as one looks back upon it, there came into use, contemporaneously with the rise of bacterial discovery and the positive knowledge that the organic solids and organic solutions are reduced by bacterial action, a diametrically opposite method of dealing with sewage, which set back the advancement of this art at least twenty years.

This curious retro-action was caused by the enthusiastic introduction of antiseptics in the disposal of organic waste. The idea of introducing processes which have the practical effect of temporarily checking bacterial life in sewage rapidly gained ground. In this way the ordinary reductions of Nature, such as we see going on everywhere under our eyes, in the fields and in the forests, were, in sewage disposal plants, temporarily stopped and postponed, with no very great apparent advantage.

Thus, along with some of the best thought in this line of work, and in the very inception of the bacterial era, came a retro-active influence in the art of sewage purification which reduced the practical value of good ideas to a small, but working, minority, and time only slowly vindicated the work of those who at an early date had positively identified innate bacterial life as the active agent in the reduction of organic wastes.

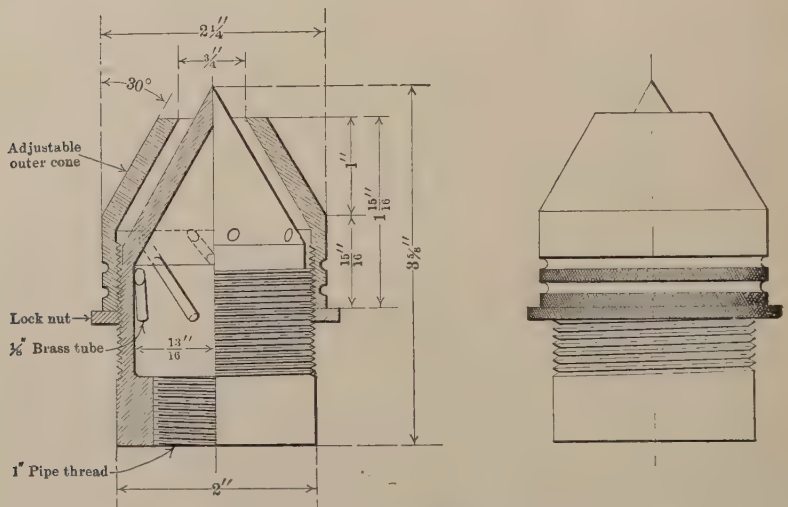
For a long time those in the minority, whose work was so little appreciated, were obliged to wait until, by the advancement of the art, the important facts which had been previously pointed out and placed on record had been generally accepted.

It may be well enough, in the common interest of science, to discover and re-discover things which have been well discovered before, but it is far better not to lose sight of any scientific advancement, however slight or insignificant, which has once been placed on record. The history of the art of sewage purification is a curious instance of where proper advancement has been made, recorded and discussed, and then, for almost a generation, lost sight of by misdirected thought.

It must always be borne in mind that there is this danger, that the majority, as a body, may lose sight of valuable ideas—ideas which lie at the foundation of all advancement.

Mr. Gregory. JOHN H. GREGORY, ASSOC. M. AM. SOC. C. E.—Mr. Whipple has given so clear a description of a sprinkling filter, as applied to the purification of sewage, that little further need be added regarding the general construction of this type; but especial attention is called to one very important consideration, namely, the distribution or application of the sewage on the surface of the filter.

In England two methods of application are in use: One by means of a movable mechanical distributor operating on a track just above the surface of the filter, and the other by a series of fixed distribution pipes with nozzles at frequent intervals through



SPRINKLING NOZZLE USED AT SEWAGE TESTING STATION, COLUMBUS, OHIO.

FIG. 1.

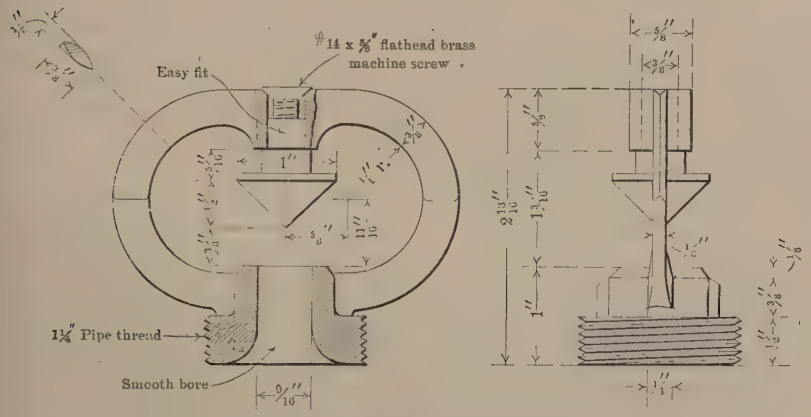
which the sewage is sprayed upon the filter, the two methods apparently giving equally satisfactory results.

It is not the speaker's intention to discuss at length the relative merits of the two methods, but it may not be amiss to say that in localities where the winters are long and cold, or where there are extended periods of cold weather, it is probable that the movable mechanical distributor would not give satisfactory results, due to trouble caused by ice and snow. Under such conditions a properly designed system of distribution pipes with sprinkling nozzles would be preferable.

In 1904 and 1905 the City of Columbus, Ohio, carried on an extensive series of experimental investigations on the purification of

sewage, at a testing station especially constructed for the purpose, and among the devices investigated were a number of sprinkling filters. Various methods of applying the sewage to the filters were tried, and, without going into details of the methods which were found unsatisfactory, it will suffice to say that a sprinkling nozzle designed under the direction of Mr. George A. Johnson, Engineer in Charge of the experimental investigations at the testing station, was finally adopted as giving the best results. This nozzle is shown in detail in Fig. 1. Mr. Gregory.

The nozzle consists of an inner 60° hollow cone, 2 in. in outside diameter, and tapped at the base for 1-in. pipe. Threaded upon this inner hollow cone is an outer 60° hollow cone, the outer cone being adjustable and held in position by a lock-nut. Eight $\frac{1}{8}$-in.



SPRINKLING NOZZLE ADOPTED FOR SEWAGE PURIFICATION WORKS, COLUMBUS, OHIO.

FIG. 2.

brass pipes pierce the shell of the inner cone, projecting inward at an angle of 45° with the vertical. The sewage is discharged through these eight pipes into the adjustable space between the inner and outer cones, leaving the outer cone in a tangential direction at the top. On leaving the nozzle, the sewage for a short distance remains in the form of a thin sheet, and then breaks into a spray of fine drops, falling upon the filter on the surface included between two concentric circles. Under normal conditions, the nozzle discharges under a head of 4 ft., measured at its base, and when thus operating 70% of the surface of the filter tributary to the nozzle (43.5 sq. ft.) was wetted by the spray in a satisfactory manner. There is, however, one serious objection to the nozzle, in that the orifices clog,

Mr. Gregory. and, although the sewage at the testing station was screened, the nozzle had to be cleaned once or twice each day.

The successful operation of the sprinkling filters at the testing station led to their adoption in the sewage purification works now under construction in Columbus. These filters will have an area of 10 acres, the sewage being sprayed through 2 108 sprinkling nozzles, spaced 15 ft. 4 in. from center to center. From the experience gained with the nozzle used at the testing station, in order that the maintenance at the permanent works might be reduced to a minimum, it was evident that a nozzle must be designed which would be free from clogging. An extended study of many types of nozzles was made, under the direction of the speaker, from which it was concluded that one having a single orifice would be the most suitable, as being least likely to clog. A series of tests was made with an experimental device, the result of which was the sprinkling nozzle shown in Fig. 2, which has been included in the contract for the permanent works.

The nozzle, which is of brass, consists of a single orifice, $\frac{9}{16}$ in. in diameter, with rounded edges, above which, held by two thin arms, is an inverted 90° cone, the axis of the cone coinciding with the axis of the orifice. The jet, on leaving the orifice, impinges against the cone and is transformed into a thin sheet, spreading out radially and breaking into a shower of fine drops. At the points where the sheet strikes the arms, the latter are ground down to sharp edges on each side, thereby permitting the sheet to heal up entirely after passing them. Under normal conditions in the permanent works, the nozzles will be required to discharge about 13.1 gal. per min. A series of experiments was made with one of these nozzles, and it was found that by varying the head from a minimum of 2.75 ft. and a discharge of 10 gal. per min. to a maximum of 5.0 ft. and a discharge of 13.5 gal. per min., 77% of the surface of the filter which will be tributary to each nozzle in the permanent works (206.6 sq. ft.) was covered in a perfectly satisfactory manner. In these figures no allowance is made for the effect of wind, but, on a large unprotected area, the effect of the wind will be such as to increase materially the percentage of surface covered. The principal advantages of the nozzle are: Its simplicity, the single orifice, freedom from clogging, small head, satisfactory spray, large percentage of area covered, and low cost.

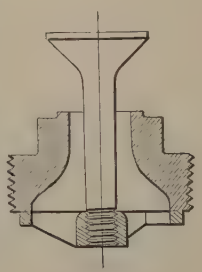
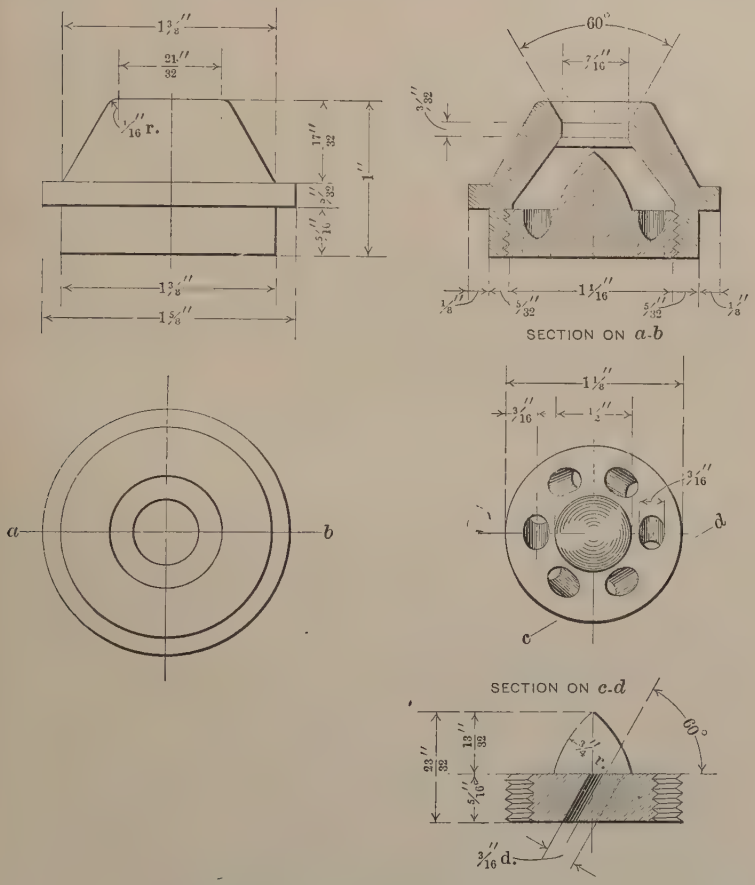


Fig. 3.

Another type of nozzle which received careful study is shown in Fig. 3. Although this type may have some advantages, the single orifice was considered preferable to an annular one, on account of its greater freedom from clogging.

In Fig. 4 is shown the nozzle used at Salford, England. As will be seen, it is in some respects similar to the one used at the testing station, and, from a few experiments, it was found that its action was very similar. Both the Salford and the testing station nozzles require a greater head than the nozzle adopted for

Mr. Gregory.



SPRINKLING NOZZLE USED AT SALFORD, ENGLAND.

FIG. 4.

the purification works, when operating under similar conditions, the spray from them, however, breaking possibly into slightly smaller drops.

The subject of sprinkling nozzles is not only of great interest, but of great importance, as the efficiency of a sprinkling filter,

Mr. Gregory

where nozzles are used for distributing the sewage, is in a large measure dependent on their satisfactory operation. The speaker is at present arranging for another series of experiments on modifications of the sprinkling nozzle adopted for the Columbus sewage purification works, and should the results warrant, he hopes to present them at some future time.

Mr. Clark.

H. W. Clark,* Esq. (by letter).—That the study of sewage purification is a fascinating one, can only be comprehended adequately by those who are actually engaged in it. Of the two branches of this study, namely, theory and practice, or why and how, sometimes one and sometimes the other plays the more prominent part in the mind of the investigator. Because of the work resulting in what Mr. Whipple rightly terms "The mist of confused notions about septic tanks, contact beds, and trickling filters," etc., which during the past ten or fifteen years has been prominent in the literature of the subject, much true information has been gained. The subject cannot possibly be studied by many men under different conditions and with different grades of sewage without conflicting ideas arising and opposing statements being made. Each honest opinion and each honest result has had its value, however, and, by comparison and elimination, the truth is being slowly made manifest. There is a certain impatience, to be sure, in the mind of the practicing engineer who is looking for absolute data upon which to base his plans, and instead of this is, perhaps, forced to compare and study many apparently conflicting results obtained by the work of many people at different places. Those chemists, bacteriologists and engineers, however, who have given months and years to experimental investigation of the subject, have an infinite patience with all apparently contradictory work, understanding full well its real difficulty. As a general rule, they are not carping critics of each other, although perhaps stoutly believing in their own theories and results, and always willing to defend them.

It is undoubtedly true that England is to-day the center of activity in the construction of sewage purification plants and of experiments upon a large scale. Many of these plants, however, are based upon principles and theories worked out in America during the past twenty years, as is willingly testified to by many English experts and engineers. This can readily be confirmed by reading the English literature on the subject. At the present day, leaving out of consideration sewage farming or broad irrigation, there are three methods by which sewage can be treated more or less successfully, namely, by intermittent sand filters, by contact filters, and by sprinkling filters. Much more work has been done in America in studying intermittent sand filtration than in studying the other two methods, and the reason, of course, is clear. The home

* Chemist, Massachusetts State Board of Health.

of sewage purification in America up to the present time has been New England, and intermittent sand filters are easily and cheaply constructed there, and, for the problems so far presented, are more satisfactory than the other kinds could possibly be. During the past ten years, however, both contact filters and trickling filters have been investigated at the Lawrence Experiment Station, and, during the past two or three years, at other experiment stations. Of these two latter methods of filtration, the trickling filter easily takes the first place. Properly constructed and operated, contact filters when built of rough material can undoubtedly give more or less efficient results, but their life between thorough cleanings is comparatively short unless the sewage applied receives good preliminary treatment, and their rate per acre is much less than that of trickling filters. Mr. Clark.

Trickling filters have been operated at Lawrence since 1899, and, indeed, the first filters at Lawrence operated in a manner similar to the modern trickling filters were Filters Nos. 15 and 16, put into operation more than fourteen years ago. As well stated by Mr. Fuller and Mr. Johnson in the Columbus report, also in the report of the engineers of the Baltimore Sewerage Commission, and as acknowledged by many English experts, these filters, constructed of stone and operated at high rates, were the forerunners of the trickling filter of the present day.

What can be done by sand filtration is fairly well known at the present time, and also about how permanent such filters are when operated under normal conditions. There is no need to quote here the review made by the writer in the report of the Lawrence Experiment Station for 1904, showing the work of many years of both experimental and municipal sand filters in the purification of sewage and in the storage of organic matter. It was evident, however, from this review that, with the experiment station sewage, about 50% of the nitrogen applied during eighteen years was oxidized; about 15% appeared in an unoxidized form in the effluents; about 35% was unaccounted for, and not more than 4 or 5% remained stored in the sand. Leaving out of consideration the upper 6 in. of material, the filters receiving this sewage were in as good condition at the end of this period as when first put into operation.

At the municipal areas in Massachusetts, most of which receive fresher sewage than that received at the experiment station, a very much larger percentage of the organic matter in the sewage is removed mechanically by settling tanks or by raking and scraping the surface of the beds, and a very much smaller percentage is oxidized than is the case with the Lawrence experimental filters. At these areas, for instance, only an average of about 25% of the

Mr. Clark. applied nitrogen is oxidized and appears in the filter effluents; about 15% appears unoxidized in the effluents, and the amount stored in the filter, removed from the surface by raking and scraping, or escaping by liberation into the air, amounts to about 60 per cent. Of this 60% by far the larger proportion is removed by settling, raking and scraping.

In this review, the writer called attention, moreover, to a fact that generally has been overlooked, namely, that the matter stored in sand filters is largely carbonaceous rather than nitrogenous. As a matter of fact, only 4 or 5% of the organic matter stored in the Lawrence sand filters after seventeen years' use was nitrogen.

There is no question of the efficiency of sand filters, neither can there be any question that the effluent produced by them is of much greater purity than can be produced by any other kind of filtration known at the present time. Neither is there any question at the present day regarding their practical permanency, nor is there any question of their cheapness of operation, considering the results obtained. Different geological and other conditions, however, require and will require at many places in America other methods of sewage purification, and for such locations the trickling filter is now the most prominent and promising method. It is fairly well known at the present time what trickling filters will do under ordinary conditions and with different depths of filtering material. With sewage as strong as that in use at the Lawrence Experiment Station, and with deep filters, rates of from 2 000 000 to 3 000 000 gal. per acre per day can be obtained, and high nitrification and the production of a non-putrescible effluent will ensue. The degree of permanency of these filters is something about which more is being learned every day, but as yet they have not been operated long enough, either in America experimentally, or abroad practically, to be absolutely certain of the length of time that they will run under different conditions before thorough cleaning will become necessary. However, the chief filters of this class at Lawrence have been in operation now for more than seven years without the accumulation of enough matter to cause trouble; in fact, the slight accumulation so far made probably has rendered them more efficient. During these seven years less than 2½% of the total organic matter applied to them, and only about 1% of the applied nitrogen, has as yet been stored. In distinction from this, contact filters at Lawrence, constructed of the same kind and grade of material, and operated at only about one-fourth the rate of these sprinkling filters, have stored, in a period of operation less than one-half as long, 24% of the total applied organic matter and 9% of the applied nitrogen and have produced very poor effluents. Contact filters of clinker and coke have been operated for many years at Law-

rence, and have given much better results than the broken-stone filters just referred to, and especially so when the sewage applied has been given an efficient preliminary treatment. Two such filters, 5 ft. in depth, have been in operation for more than six years, and at the present time receive sewage at the rate of a little less than 500 000 gal. per acre per day. Mr. Clark.

When the effluents of trickling or contact filters are compared with the effluents of sand filters, the real difference is indeed shown. The effluent of the average sand filter is generally clear, free from turbidity, odorless, and often can fairly be called sparkling. The effluents of the best contact and trickling filters, on the other hand, are generally more or less turbid and unattractive. Such effluents as are given by the best of these filters at Lawrence, however, are non-putrescible; even the effluent of a poorer filter becoming so when the matters in suspension are removed by sedimentation. Such effluents improve rather than deteriorate on keeping, and when mixed with even an equal volume of ordinary river water containing much dissolved oxygen, they improve rapidly.

In order to lessen the area necessary for the treatment of a given volume of sewage, and to improve the final effluent, preliminary treatment deserves great attention. It seems possible that screens and methods of screening may be greatly improved, and, on account of this, a more economical and efficient method be devised for removing suspended matters than settling and septic tanks, or at least be so improved that less will be left for such tanks to do. Septic tanks for the treatment of the sludge, only, after ordinary sedimentation, spoken of by Mr. Whipple as a process now being considered by English investigators, have been in use at Lawrence for the past seven years, being first described in the Lawrence report for 1899.

Finally, sand filtration is an ideal finishing process. A complete plant at the Lawrence Station, which has been in operation several years, is as follows: (1) tank treatment; (2) sprinkling filters; (3) sedimentation; and (4) sand filtration. The rate at which such a plant can be operated depends upon the grade of sewage to be purified, but, at Lawrence, rates of 2 500 000 gal. per acre for the sprinkling filters and 500 000 gal. per acre for the final sand filters can be easily maintained, with the production of a clear and highly nitrified final effluent. Such a plant has recently been recommended by the engineers of the Baltimore Sewerage Commission for the purification of Baltimore sewage.

While experiment stations can furnish reliable data as to methods of filtration, rates of filtration, and degrees of purification, they cannot, of course, give illustrations of engineering skill in design and construction, such as the works abroad present. Mr.

Mr. Clark. Whipple's discussion is an interesting description of such works, and the English engineering journals have been full of such descriptions for a number of years. Adequate and reliable analytical data are now needed, in order to show what these purification plants actually accomplish—such data as have been given out from Lawrence, Columbus, and elsewhere in America.

Mr. Winslow. C-E. A. WINSLOW,* ESQ. (by letter).—Mr. Whipple has covered the ground so thoroughly that little remains to be done except to congratulate him, and to express appreciation of the importance of so clear, complete and exhaustive an analysis of the fundamental principles of sewage treatment.

What strikes the writer most forcibly in Mr. Whipple's address is his frank recognition of the great complexity of the problem. We no longer hope, except in rare cases, to treat sewage by dumping in chemicals and discharging the supernatant liquid into the nearest stream, or by pouring it on sand beds without preliminary treatment or subsequent sludge removal. Scientific sewage treatment involves a number of distinct technical processes, each of which must be carefully regulated in order to secure the best results. The disposal of the sewage from the South Metropolitan District of Boston, for example, would require every year the conversion of 3 000 tons of nitrogen into a stable form, a problem in industrial chemistry of no mean magnitude.

There are two main phases of the sewage problem which deserve special consideration: the oxidation of organic matter and the removal of suspended solids, the first being a chemical change accomplished by bacterial action, and the second a physical change generally brought about by mechanical means. The writer believes that these two separate problems must be faced in any sewage disposal project which aims permanently to produce a clear and stable effluent. It was at one time held that, in the intermittent filtration of sewage through sand, no sludge was produced. The experience of nearly 20 years in Massachusetts has shown quite clearly that this is fallacious. At Brockton, for example, nearly 6 tons of sludge are removed from the beds for every 1 000 000 gal. of sewage filtered: and even with constant care the beds which have been in use for 10 years or more are becoming so clogged below the surface as to impair their operation seriously.

The fact should be faced that a certain quantity of solid sludge must be produced by any efficient process of sewage treatment. This sludge, however, may be reasonable in amount and comparatively innocuous in character, if the various parts of the system be properly adjusted; and the best results will generally follow

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from a division into preparatory processes, purification processes and finishing processes such as Mr. Whipple has outlined so clearly. Mr. Winslow.

The central part of the whole system is the purification process which always consists in the conversion of unstable organic matter into some stable form: nitrates, gases, or humus-like organic compounds. Of the three chief methods of cultivating the bacteria which carry out these changes, either may prove most satisfactory in a given case. Intermittent filtration yields the best effluent, and is the cheapest method where the requisite sand areas are available. Contact beds, as Mr. Whipple has pointed out, are especially indicated where only a limited head is available, although the writer questions somewhat the statement that, in small installations, they can be made to operate with comparatively little attention. He is inclined to think that a trickling filter, properly designed, would be less seriously deranged by neglect than are many contact beds which he has seen.

For most purposes the trickling filter offers the most satisfactory method of securing an oxidized effluent. For nearly a year Boston sewage has been treated at the Technology Experiment Station on a trickling bed having an area of 200 sq. ft., and at a rate of 2 000 000 gal. per acre per day, with excellent results. The effluent is scarcely improved in appearance or in its organic nitrogen content, and sometimes contains more suspended matter than the applied sewage. It is uniformly stable, however, and the fact that it contains so much suspended matter is the guaranty of the life of the filter. Beds of this type have the great advantages of operating at a much higher rate, and with a greater freedom from clogging, than any other form. The great practical point to be worked out in their operation is the best system of distribution, as their success depends mainly on the fineness and evenness with which the sewage can be spread over their surfaces.

Before treating Boston sewage on the trickling bed, it has been found best to remove a portion of the suspended solids by two successive preliminary treatments. The sewage treated at the Experiment Station first passes through a detritus tank containing a screen with $\frac{1}{2}$-in. openings. Here the floating matter and gravel are removed. The total quantity of material taken out in this tank amounts to only 0.65 cu. yd. per 1 000 000 gal., and it is so largely inorganic as to be inoffensive when spread on the surface of the ground. Next, for the best results, the sewage is passed through septic tanks. The writer is inclined to agree with Mr. Whipple that the beneficial effect of the septic tank in relation to the sewage is simply the effect of physical settling, the only advantage of the septic process over plain sedimentation lying in the subsequent decomposition of part of the sludge. Mr. Whipple's suggestion of

Mr. Winslow. limiting the septic action to the sludge alone, along the line of the Hampton experiments, appears to be a logical conclusion from this position. The experiments made at Brockton a few years ago by Mr. Bolling indicated, however, that a septic tank treating concentrated sludge fills up very rapidly. The conditions which make liquefaction rapid and successful in one septic tank, and slow and inadequate in another, are very little understood and should be studied exhaustively. In the tanks of the Technology Experiment Station the decomposition of sludge appears to be unusually active, three-quarters of the nitrogen deposited as albuminoid ammonia being destroyed. At the same time, Boston sewage readily shows the bad effect upon the effluent of a too prolonged septic action. Ill-smelling gases (hydrogen sulphide, mercaptans, etc.) are formed from the sulphates introduced by the leakage of sea-water, and the subsequent processes of purification are interfered with unless the effluent be well aerated. Therefore, the period of passage through the septic tanks has been reduced to 7 hours, so as to minimize the chemical changes in the supernatant liquid as much as possible.

The experimental septic tanks remove about half the suspended matter from Boston sewage, but in passing the effluent through the trickling beds no further reduction takes place, and, at times, there is an actual increase. In order to produce a clear effluent, it is necessary that some finishing process should follow the trickling filter; and a brief sedimentation lasting about 2 hours is now being used. The average results for the first 3 months of 1906 showed 126 parts per million of suspended solids in the crude sewage. Septic treatment reduced this figure to 60, and, for this particular period, the trickling filter increased it again to 84. Sedimentation of the trickling effluent finally decreased the suspended solids to 40 parts. If still better results are desired, straining or chemical treatment might be tried. The writer's colleague, Mr. Phelps, is at present studying the use of lime for the production of a clear and sterile final effluent. The separation of solids may be carried to any desired point of perfection, if once the essential oxidation of organic matter has been carried out in the trickling bed.

The main point in this discussion seems to be its emphasis on the fact that sewage disposal is an engineering problem which is always difficult, and which varies in each individual case. Only expert study of local conditions, the character of the sewage to be treated, the grade of effluent desired, the filtering material, the land area, and the head available, can solve it properly.

Mr. Whipple. G. C. Whipple, Assoc. M. Am. Soc. C. E. (by letter).—The writer has been pleased to observe the interest which has been shown in the discussion of this subject. The remarks of most of

the discussors have been so much in accord with what was said in the opening discussion that there is little occasion for adding any concluding words. Mr. Whipple.

Many seem to be interested in the evolution and development of sewage disposal methods. This is certainly an attractive phase of the subject. It is undoubtedly true, as mentioned by Mr. Alvord and others, that certain processes have been used, dropped, and taken up again. This, however, does not indicate a reactionary course of events, for whenever old ideas have been taken up again it has been because new light has been thrown upon them by some other new discovery. That a steady advance is being made in the practical solution of the problem seems to be unquestioned. Experimental investigations have done much to contribute to this advance. To a great extent, the new ideas have originated with those in charge of such investigations, but in many cases practice has come first and theory afterward.

Messrs. Johnson and Clark, in their discussions, have emphasized the importance of intermittent sand filtration. The writer agrees with them that, under the conditions which exist in New England and in certain other parts of the United States, this method is the most satisfactory one that can be used. Little attention was given to this matter in the opening discussion, for, as the speaker stated, he felt that this subject was so well known to American engineers that there was no need of enlarging upon it. It is well, however, that it should not by any chance be placed in a wrong perspective, and the writer is glad that this method has been emphasized in the later discussions. Nevertheless, the fact should not be lost sight of that there are vast stretches of country in the United States, notably in the South and West, where this method of sewage treatment is impracticable, and where some of the newer processes must be used.

Attention has been called to the fact that with intermittent filtration it is not customary to use preliminary settlement in septic tanks, and therefore that the figure for "population per acre," as given in Table 1, unfairly represents the possibilities of this method. Strictly speaking, the point is well taken, but the writer believes that the figures given may be considered as fairly representative of what may be obtained by the various methods as they are practically used.

Mr. Maignen has opened up several questions which might be discussed at length. The manurial value of sewage sludge has been a subject of inquiry for many years. At Preston, England, some practical experiments were made in connection with this subject, and it was learned that speaking generally the manurial value of sludge was practically equal to that of ordinary manure. It was

Mr. Whipple. found that the advantage to be gained by applying either manure or sewage sludge to growing crops yearly decreased after the first application: During the first year, hay crops were increased by applying manure sludge; in the second year, the advantage was less noticeable; in the third year, it was still less, and so on. In other words, cultivated fields are not improved by over-manuring, as agriculturists know. This fact probably helps to explain the difficulty of disposing of sludge to farmers after sewage works have been in operation a few years. There seems to be little difference in the manurial value of the sludge from a septic tank and that from a plain settling tank. At Birmingham it has been found that crops will not grow in septic sludge unless soil is mixed with it.

Regarding the cost of sewage treatment in England, the writer, when there, collected numerous data, but as they were so conflicting, and as the English conditions are so different from those existing in America, and, furthermore, as the data were confidentially furnished in many cases, he does not think it best to make them public at this time. It is understood, however, that the forthcoming report of the Royal Commission on Sewage Disposal will contain data of the cost of sewage disposal which will be more accurate and complete than any figures which the writer might furnish.

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TRANSACTIONS.

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ENGINEERING EDUCATION.

An Informal Discussion at the Annual Convention, June 27th, 1906.

SUBJECTS FOR DISCUSSION.

"What is the Best Preparatory Education for the Civil Engineering Profession?"

"Is Technical Training the Best Education for Executive Work?"

BY MESSRS. GEORGE F. SWAIN, HAROLD BOUTON, GEORGE W. RAFTER, EDGAR MARBURG, GARDNER S. WILLIAMS, OLIN H. LANDRETH, BERNARD R. GREEN, EDWIN F. WENDT, A. N. TALBOT, F. R. COATES, BASSETT JONES, JR., AND GEORGE F. SWAIN.

Mr. Swain.

GEORGE F. SWAIN, M. AM. SOC. C. E. (by letter).—Most people will admit to-day that civil engineering, like other branches of engineering, belongs to the learned professions, and should require a preliminary technical training corresponding generally to that necessary for the lawyer or the physician. There is much to be said for the old apprentice system of becoming an engineer, or for the method by which a young man enters an engineer's office after an ordinary public school education, and slowly works his way along, studying, as he learns, by doing, the practical details of the profession. Engineering is more—much more—common sense and "gumption" than it is science, and the school cannot make up for a lack of these qualities. Many of our best and most deservedly

Mr. Swain. eminent engineers are men who, by force of character and perseverance, have worked themselves up in this way. Yet these men would probably be the first to acknowledge the great advantage which a proper technical education would have been to them, and they would not consider for a moment bringing up their sons in the way which they followed. Yet it is probable that they fail to realize the benefits which they derived from the stern discipline of actual life, and from the necessity which they were under of making up for lack of opportunity by hard work and diligent application. The trouble with the young man, between the ages of sixteen and twenty-two, who is given the opportunity of a higher education, is that he fails to realize his opportunity, and does not take advantage of it; and, as a result, many of the graduates of technical schools and colleges have neither accurate knowledge of any one subject, nor the ability to think clearly and logically, nor the power of taking up a new subject and mastering its fundamental principles without assistance.

"I would set all the young men to work," said Socrates, "and send all the old men to school." And this, while of course impracticable, involves a deep and fundamental truth, namely, that we learn mostly by experience, rather than by precept, and that only as we approach middle life do we awaken to the advantages of thorough training and accurate knowledge. Experience is the greatest of all teachers, but is an expensive one. The great difficulty that confronts the teacher is to awaken the student to a sense of his responsibility and his opportunity, to make him patient and even anxious of correction, and to make him see that the true object of his education is to train himself to accurate thinking, to high ideals, and to a proper balance of all his faculties, so that he may make of himself the best that is possible. As Dr. Munger has said, "Education is to teach us how to live, not how to make a living."

It is undoubtedly true, however, that most young men who go to a professional school with a proper sense of the opportunity, and embrace it earnestly, will get from the course what they could not, or at all events would not, get without it. But the technical school cannot make an engineer; it can only give the opportunity for the young man to acquire a training, an independence of mind, a character, which will make him first of all a man, and show him how to live, and further, to acquire a familiarity with the fundamental principles of science, which he ought to know in order to be an intelligent engineer instead of a parrot, an imitator, a rule-of-thumb man. The degree does not and cannot make the engineer, though some schools apparently deceive themselves by thinking that it does. The school cannot even teach; it can only offer

opportunities for a man to learn, for nothing is of real value in this world but what we gain by our own efforts. The college is not a restaurant, where young men come to be filled, but a gymnasium, where opportunities are offered for the development of all the faculties, not only mental, but physical and moral. It is self-evident that a development of this kind is the proper and necessary training for the highest success in any profession, and while such development can in some instances be obtained independent of the school, yet a school which offers suitable opportunities ought to give what most men would not be apt to obtain if left to themselves. After all, we are creatures of habit, and habits acquired in early life, whether physical, mental, or moral, are apt to remain and if bad are hard to eradicate; hence the importance of acquiring proper habits of thought, as well as of action, as early as possible. Mr. Swain.

Assuming, then, as a fundamental proposition, that a technical education is the proper preparation for the civil engineering profession, and that few men have the character, the courage, and the perseverance, to be able to develop themselves unaided, the question remains, what should be the character of that education, and how far it should go. We confront once more the old question, which so many able minds have discussed. "What knowledge is of most worth?" In considering this question, with reference to engineering education, one must never forget that the aim here, as in all other education, should be first of all to make men. Engineering education must not be narrow, must not be confined to strictly professional subjects, but must be broad enough to develop the man on all sides. At the same time, we must remember one of the fundamental principles of the modern education, which is, that of two subjects which will give equally good training, the more useful one should be chosen. There is no excuse, at the present day, for teaching subjects which will be of no possible use to a man in his professional or social relations, simply because they afford good mental training, when there are many useful subjects which, if properly taught, afford just as good or even better training. The interest of the student will lie with the subject for which he can see a use, and interest is a necessary factor in education. But even the useful subjects cannot all be taught in a technical course of the usual length, or even in a college course followed by a professional course. Some selection must be made. And here it will be well to bear in mind that, as President Eliot once said, "the actual problem is not what to teach, but how to teach." We must not endeavor to teach everything which a man will need, but must make a judicious selection of subjects, and teach these thoroughly, in such a way that the student will gain the power to take up and master new subjects by himself. And here is reached one

Mr. Swain. of the fundamental and greatest defects of education at the present day. Too much attention is devoted to the question, what shall be taught, and too little, or sometimes almost none at all, to the question, how it should be taught. Much time is devoted to the arrangement of the curriculum, and then an important subject is assigned to a teacher who can neither interest the students nor make them understand it; or who, perhaps, instead of training them to think, and giving them in this way some power of doing things they have never done before, simply turns his class-room into a restaurant, and fills his pupils up with facts, the bearings of which they are unable to appreciate, and which they promptly forget. Comparatively little attention is paid to the appointment of teachers, it being assumed, apparently, that if a man understands a subject, or appears to understand it, he can teach it to others. It is not made a requisite for the teacher's post that a man shall have been trained in pedagogics, that he shall know something of the theory and history of education, or of psychology; these things are neglected, and men are often appointed to high positions as teachers who have had no training in education, who have to learn that science as they would any other, and meanwhile, at the expense of their pupils. Good teachers are extremely rare, and the faculty for teaching is a gift, perhaps to a greater extent than most faculties. With some men it is almost intuitive, and such men do not require much training. But with most men it has to be learned. Too many men teach because they could not succeed in practical business life, and, as a matter of fact, many of them cannot teach efficiently. Ask the best men in a class from any of our colleges or professional schools, and they will probably agree in telling you of important courses from which, through no fault of their own, they derived neither information, nor training, nor inspiration. It is a great pity that the results of education cannot be quantitatively expressed, and shown at the end of the year, in a balance sheet, in dollars and cents; and, further, that trustees and faculties are not dependent for their income upon the results of such a balance sheet. To pursue this line of thought would carry us too far, but the writer's advice, to the young man who wishes to study any subject whatever, is to go to the institution where there are the best teachers of that subject. Material equipment, laboratories, and so forth, are of no consequence in comparison. Mark Hopkins at one end of a log and a student at the other make a university. The teachers of engineering should study, not only engineering, but teaching, and should study the latter more than the former. One of the most encouraging educational steps in recent years in America has been the formation of the Society for the Promotion of Engineering Education. The profession of teaching is one of the most

important to the community, but it is not recognized as it ought to be, and it is not paid as it ought to be. Some universities, even, expect to obtain men competent to teach all branches of civil engineering and to occupy positions where they should be inspirers of young men to mental achievements and to high moral ideals, and all for $1 200 a year. And even the American Society of Civil Engineers does its part, the writer regrets very much to say, to keep down and render unattractive the profession of teaching, by refusing to recognize the work of a teacher in charge of a department of engineering as professional experience in charge of work. This Society has preferred to recognize, as eligible to full membership, the man who has had a few years' experience in giving lines and grades for sewers, or superintending simple practical engineering works, in preference to the man who is in charge of an engineering department of a technical school, and devotes himself to teaching and inspiring the future members and leaders of the profession. In the writer's opinion, this Society can do no better work, to raise the dignity of engineering teaching and of the engineering profession, than to remove this restriction from its Constitution. It will gain much, and lose nothing, by recognizing experience in teaching as equal to experience in practice as a requisite for membership. Mr. Swain.

The curriculum of an engineering course should be almost entirely prescribed, with few optional or elective studies. The main opportunity for election should be between different lines of study, as for instance between civil engineering, mechanical engineering, electrical engineering, etc., but the line or aim once chosen, the course should be laid out by competent teachers, and should be prescribed. The idea that students should be obliged to undertake difficult and unpleasant tasks simply because they are difficult and unpleasant is happily outgrown, but life is full of such tasks, and one of the most useful results of a proper system of training is the ability to do drudgery and to attack and surmount difficult tasks cheerfully and successfully. One of the dangers of a too literal and extreme application of the elective system is that, improperly understood, it cultivates a tendency to shirk difficulties and a disposition to avoid unpleasant tasks.

One more point is to be considered. If the desirability of breadth and utility is granted, the question is, whether they should be arrived at simultaneously or successively. It is held by many that they should be attained successively; that the public school or secondary school, which gives a boy his preparatory education, should be followed by the college, which gives him his broad general education, and this by the professional school, which gives him his technical education. This is the zone theory of education. The

Mr. Swain. difficulty with this arrangement is that during his college course the student is working less earnestly and with a less definite aim in view than if he were pursuing a professional course from the beginning. Every institution of learning should be looked upon primarily as a place where young men and women go to do hard work; and it should be impossible for anybody to remain in the institution who does not come in this spirit and conform to this requirement. Healthful and wholesome amusements, recreation and exercises should be a part of the training, but secondary to the main objects in view. A wrong attitude toward college work prevails to a great extent, not only among young men, but among their parents as well. It is unfortunately true that in most or at least in many colleges numbers of young men dawdle away their time, doing little or nothing except to cram for examinations; and that they emerge with little except a diploma and an enlarged cranium, and, perhaps, with a bad habit or two. This should be made impossible. The writer has never been connected with a college, but he knows from long experience how difficult it is to make even students in a professional school awaken to a due sense of their opportunities, and of the proper relations of things; and it must be much more difficult in a college, where a larger proportion of students are browsing around without any definite aim, and having a good time incidentally, giving no serious consideration, before graduating, to the question of their future careers. If this view is correct, it would seem to follow that the student should be urged to select as early as possible his professional course, at least within broad limits, and that during his college course, if he takes one, he should lay out his studies with distinct reference to his future professional course. It would also seem that some non-technical subjects should be carried on, even in the professional school, so that interest in such studies should not be entirely lost. Change of occupation is rest, and studies of different kinds, as for instance history and mathematics, may each prove a recreation to a mind wearied with the other. Such an arrangement as has been outlined is really a continuous professional course from the beginning, in which the proportion of professional studies increases in each year; it allows the student to work always with a definite aim in view, and at the same time the proportion of technical work in the early years is not so great as to preclude a change of course if the student comes to feel, as he advances, that some other branch of professional work than the one first selected seems to be better fitted for his capacities.

Summing up the preceding discussion, the following are the principles which the writer would lay down in answer to the question which has been propounded:

1.—A technical education as given in our civil engineering schools is, if properly appreciated and made use of, the best preparation for the practice of the profession; and at the present day it is almost a necessity. Mr. Swain.

2.—In laying out an engineering course, the aim should be, first of all, to develop broad-minded men, who can observe correctly, reason logically, express themselves in language and on paper, men with imagination and with character, and with good physical development.

3.—Useful subjects of study, which admit and require training in thinking, should be preferred to studies which are mere accomplishments.

4.—Studies which involve discipline of the mind and observation should be preferred to those which merely give information.

5.—Subject to the above restrictions, what is taught is not as important as how it is taught. The teaching profession should be better paid and made more attractive to able men.

6.—The curriculum leading to any of the engineering professions should be almost entirely prescribed.

7.—The choice of a profession should be made as early as practicable, and a continuous course should be arranged with that profession in view, from the beginning of the higher education. A course of five years, or perhaps of six years, either in one institution or in two, seems to be desirable for a thorough preparation.

8.—The American Society of Civil Engineers should recognize the dignity of the teaching profession by making experience in teaching equal to experience in practice as a requirement for membership.

Passing now to a consideration of the question "Is Technical Training the Best Education for Executive Work?" the writer's reply would depend upon the character of the technical training, that is, upon what is taught, and more especially upon how it is taught. It is a common criticism that graduates of technical schools are narrow, and that, while suited for subordinate positions, they are not as well qualified for high administrative positions as college men. The writer believes that, taken broadly, this criticism is unsound—that is to say, he does not believe that the average college man is better fitted for administrative work, or is any broader, than the average technical graduate. Nevertheless, he believes that there is much suggestiveness in the charge, and that the technical schools may profit by considering it. The number of college graduates is very much greater than the number of technical graduates, and probably a larger proportion of the latter are from humble homes, where they have been denied social advantages, and are lacking in polish, and perhaps in good manners.

Mr. Swain. They have gone to the technical school because they knew that they would have to earn their own living, for which they were obliged to prepare themselves as quickly as possible, and they have, by inheritance as well as by force of circumstances, a tendency to take an interest only in the practical professional work, and to give little attention to acquiring breadth of interest or comprehensiveness of view. They have looked at everything from the professional standpoint, and with a magnifying glass, and they lack the mobility of mind that would enable them to take in a problem or a condition in its entire scope, or to appreciate all the various sides of a question. Such an attitude is not that of the successful administrator. The man who looks at a thing through a microscope sees more—but he also sees less—than the man who looks with the unaided eye; and the tendency in any detailed study of a scientific or technical problem is to concentrate so much attention on the details that the general relations are not perceived. The engineering student is constantly under this temptation, and unless it is counteracted by good teaching, it may soon become a habit. Unfortunately, here again many teachers fail to do what they should, being narrow themselves, or lacking in a knowledge of the large practical relations of the subjects which they teach. Engineering students constantly seem to the writer to be like men studying a book with a microscope; who can tell the details of each particular portion of the work, but who have failed to see what it was all about.

It is not apparent to the writer, however, that engineering studies are very different from other studies in a narrowing tendency. How many students of history, for instance, arrive at correct general conclusions or accurate ideas regarding the general tendencies of a period? Here, as in the study of science, there seems to be about equal opportunity for spending so much time upon detail that general relations are obscured.

A certain largeness of vision is essential for administrative work, but, in order to judge correctly as to the relative value of technical and general education as a preparation, we must, not only consider what technical education can be, and ought to be, but, instead of comparing college graduates with technical graduates, we must consider whether the same man would be better fitted by the college course or by the technical course. In doing this, the following are some of the elements to be considered:

1.—Most executive or administrative work has to do with engineering, or involves engineering as an important element. Our railroads, mines, manufacturing establishments, etc., depend upon applied science, that is, upon some branch of engineering. A knowledge of engineering, therefore, if it is accompanied by breadth of

view, largeness of conception, and the personal qualities necessary, must be of great advantage in rendering the administrative officer able to form his own opinions, and in enabling him to direct the energies of his staff in the directions most productive of efficiency, economy, and industrial development. Mr Swain.

2.—Scientific study certainly has the great moral advantage of training men to search for truth, to keep their minds open until a result is obtained, and to be satisfied with no makeshifts or evasions. Such an attitude of mind must be consonant with the highest kind of administration, however much it may conflict with the necessities of politics, graft, or deception, which seem to be the ruling elements in some executive positions.

3.—Scientific study and the pursuit of truth for its own sake conduces to honesty, both of mind and of action, to frankness and fearlessness, and to uprightness of purpose. However narrow engineers may be, the writer believes that for these qualities just mentioned they are not excelled by the members of any other profession.

4.—Technical training, and particularly technical experience in the handling of men, is clearly of advantage in any executive position.

After a consideration of these elements, the writer—while ready to admit the fact that many men technically trained lack the breadth of view and adaptability which is essential in executive work—believes nevertheless that technical training, if the course of study is properly laid out, with a proper proportion of liberalizing studies, and pursued under teachers who direct the students always toward the larger view, is the best preparation for executive work.

Harold Bouton, Assoc. Am. Soc. C. E. (by letter).—The question in regard to the best preparatory education for the civil engineer deserves the interest and very earnest thought of every member of this Society. Mr. Bouton.

The subject is of very great importance, and should be accorded more than a casual or indifferent discussion. Some radical measures should be undertaken to change the conditions which are responsible for the inability of a layman to distinguish between a qualified engineer and a fraud or an incompetent. Incompetency should be reduced to a minimum. Fraud should be eliminated.

The very question seems to infer that some preparatory education is necessary, unless the writer is mistaken in the import of the word "education" as used, and that experience alone will satisfy its definition. But, at the present time, who can practice as a civil engineer? Anybody; anyone, of any age, character or manner of education. With himself as the sole judge of his own qualifications, he has only to claim this distinction in order to set up its practice.

Mr. Bouton. As a matter of fact, who do practice? They may be divided into four classes:

1.—Frauds;
2.—Those depending on practical experience alone;
3.—Those ambitious enough to have prepared themselves as well as circumstances, wisdom and capacity permitted;
4.—Those who have taken technical courses under the instruction and supervision of others.

The frauds have been confined to the first class. The incompetent are not confined to any. They may be found in all four.

A few results of this condition of indiscriminate freedom of practice are:

1.—Lack of respect, if not contempt, by laymen, for the accuracy, ability and responsibility of the profession at large;
2.—Loss of its dignity as a profession;
3.—No recognized code of ethics;
4.—Contempt for a technical or preparatory education by those brilliant geniuses who know by instinct or natural aptitude and without study the rules and laws of mathematics and Nature;
5.—No standard basic course, defined for those who desire preparation, but who have to undertake it without the assistance of the instructor;
6.—The graduation from technical courses of at least fifty-seven varieties of civil engineer; most of them specialists;
7.—The ease with which some degrees can be procured;
8.—Lack of protection for those who blindly undertake a technical course of which they can know nothing more than that it will graduate them as C. E., if they will comply with its requirements;
9.—Loss of time and money to ambitious students in technical courses without adequate return;
10.—Disregard for the best methods of economic engineering design, practice and construction;
11.—Disregard for the laws, circumstances, and conditions, governing the locality and project;
12.—Faith in handbooks as the gospel of engineering;
13.—Ignorant, dangerous and extravagant design;
14.—Fraudulent, illegal, unjust, indefinite, and inadequate specifications and contracts;
15.—Excessive cost to employers;

16.—Low grade of construction to prevent excessive cost to contractors; Mr. Bouton.
17.—Innumerable law-suits;
18.—Endangering of the health, safety, and interests of the public.

Some of the causes responsible for the foregoing conditions are:

1.—Entire lack of any regulation by Federal or State governments, or this Society, of who shall practice;
2.—Lack of requirement for education of any kind;
3.—Lack of minimum requirement of preparation of all who desire to practice;
4.—Lack of uniformity and practicability in technical courses, as to time required, methods and subjects of study;
5.—Lack of intelligent application of theory to practice;
6.—Incompetent instructors;
7.—Specialization of technical courses with insufficient foundation of the primary principles;
8.—The present policy of this Society, that experience is alone sufficient without any preparatory education whatsoever, for admission to all grades;
9.—And that the experience of an axeman during his seventeenth and eighteenth years is accepted as an equivalent for graduation from a recognized school of engineering for admission as Junior;
10.—Indifference of the profession at large to the deplorable conditions.

The public deserves better protection. The dignity of a responsible profession involving the most nearly exact sciences, requires it.

Is it feasible to change this state of affairs? It has proven so in law and medicine, neither of which can be made as exact as engineering. The Federal and many State governments require a test satisfying various civil service regulations of those entering the public service.

Law and medicine, in the majority of instances, deal with the safety and affairs of an individual or a few. Engineering deals mostly with those of a multitude.

Statutory law is as various as the local necessities, customs or policies of the States and Territories. Its specialties are only limited in number by the varying phases of human affairs and intercourse. This has not prevented the almost universal requirement that there shall be a minimum preparation upon which to base the

Mr. Bouton. practice of law. On the contrary, the general sentiment and tendency is for a higher standard.

In every civilized country certain essential preparation is required of those who desire to practice medicine or surgery. Public policy has necessitated a license, in order to eliminate or limit the quack. But that profession itself has done much to merit the confidence necessarily reposed in it. The annual convention of the American Medical Association, held this same year and month in Boston, advocates a higher standard than ever before, and a better supervision of professional schools.

The architects, whose profession most nearly resembles Engineering, have examined the question. At present it is said that the States of New Jersey, Illinois and California require a license for the practice of architecture.

In New York State, in 1901, agitation in this regard resulted in a meeting being called by the Architectural League of New York and a preliminary committee being appointed to find with what favor the profession at large would consider a bill drawn for presentation to the State Legislature to institute a compulsory registration of architects.

"Circulars asking for the expression of opinion for or against the proposed legislation were sent to 995 architects, and elicited 520 replies, classified as follows: In favor of legislation, 493; against legislation, 20; not in favor of draft of bill as issued, or otherwise doubtful, 7. The Buffalo, Brooklyn and Central New York chapters of the American Institute of Architects endorsed the legislation proposed."

The movement failed.

"The New York chapter of the American Institute of Architects put itself on record as opposed to any legislation (at that time). Its reasons were briefly:

"No law should be urged unless, if enacted, the result will be positive and beneficial. No law can possibly be enforced at present which will take away from any individual the right to design and construct his own building, or to have this work done by anyone whom he may select, even though he be not a registered architect. The apparent analogy with the bar and medical profession does not exist, as only members of the bar and regular physicians are allowed to practice in this State; and the registered architect will have an empty title with no powers or rights attached. No examination can deal with other than constructive knowledge and among the registered architects will certainly occur many names which one hestitates to recognize as properly entitled to be called architects. The anomaly will be presented of architects registered by the State on account of presumed constructive knowledge, while the engineers, whose special attribute is such knowledge, are not registered. Architects properly desire to be considered members of the artistic fra-

ternity, rather assimilated to painters and sculptors than to strictly professional men, as lawyers and physicians, or to craftsmen, as builders and plumbers; and this position will not be advanced by State registration. Architectural societies, and especially the American Institute of Architects, furnish the best means for conferring deserved distinction on the members of the profession; a State license will not confer such distinction."* Mr. Bouton.

How much strength there is to these objections, generally, it is no part of this discussion to argue. As to the alleged illegality, no further comment need be made than to suggest the inherent police power of a State to regulate whatever may affect the health and safety of its citizens; and of which power the building codes of different localities are examples. In addition, it may be said, of the claim somewhere made, that where engineering problems are presented or involved, it is customary to call in an engineer, leaving the artistic side to the architect. Were this done, which it is well known is not the general custom, it means calling in any engineer, with the consequent chance of competency. But, one will say, the local building code must be complied with. True, but who draws that? Any engineer or architect, or perhaps political favorite, who has sometime built a cellar wall.

The practicability and benefits, as shown in those professions which have installed it, would seem to insure its success in engineering.

The following would do much toward remedying the evils which at present invest the profession and its practice:

1.—Registration of all; present reputable and recognized engineers to be exempt from any test;
2.—Licensing all who desire to enter practice in the future;
3.—Minimum standard requirement of preparatory education to be the basis of such license;
4.—Supervision of, or recommendation to, technical schools;
5.—Delegation of authority by the State to the Society to regulate and enforce such registration, licensing and supervision;
6.—Enlargement of the Constitution of the Society to comprehend such action;
7.—Uniform technical courses of a modern and practical kind;
8.—Uniform entrance examinations;
9.—Change of the Society's Constitution to set the example and to exert the influence that would naturally result;

* The article from which the above is taken, and the draft of the proposed bill, may be found in *The Engineering Record*, January 26th, 1901, p. 88.

Mr. Bouton.

10.—Schools and reputable engineers, in or out of the Society, should join to place the profession on a healthy basis.

This Society, the foremost technical society in America, should take the initiative. It should appoint a committee to investigate the present methods and conditions of preparatory education and qualifications for the practice of civil engineering in America, and report to the Society the result of its examination, with such recommendations for improvement as it deems advisable, with a view to the taking of such further steps by the Society as may be necessary and feasible to regulate and limit the practice of civil engineering to those qualified by preparation, competency, responsibility and character, and to elevate the profession to as high a standard as possible.

Mr. Rafter.

GEORGE W. RAFTER, M. AM. SOC. C. E. (by letter).—This question is a broad one, and opens up the entire field of present practice in technical schools. It seems to the writer that the best preparatory education for the civil engineering profession is that which best prepares for the profession, but the difficulty in this statement of the case is to know what does best prepare. Individual idiosyncrasy is so important that what is best for one person may not be best for another, and consequently the personal judgment of the instructor must come in to decide what is best for each particular case.

From this point of view, large schools are a mistake—indeed, if engineering schools did not number more than 100 or 200 pupils, assuming that the class of instructors was up to the mark, there would be a distinct gain. At the large schools, there is so little personal contact between instructor and instructed that the student is practically on his own resources, and if he has within him that which makes an engineer anyhow, there is really, for him, very little value in the technical training—the most that can be said is that he may get to work sooner, which is a much less important point than is generally assumed. A man's work is not measured by years, but by quality, and there are many men whose important work has all been compassed within a few years.

The writer supposes that it is within the experience of everybody who has studied at a technical school that, during the first year or two of practical work, he is chiefly engaged in unlearning the things learned there, and during the next year or two he mostly forgets the things learned—at any rate, so far as mere memorizing is concerned. There remains, therefore, for the average student nothing but the training; and this question really narrows itself to, What is the best method of broadening the mind so that one attains comprehension? We return, therefore, to the original propo-

sition, that the best preparatory education for the civil engineering profession is that which best prepares for the profession, that the best education for one individual may not be the best for another, and that, therefore, engineering schools should number only a few students who, under the personal supervision of capable instructors, would receive that which best suited each individual's needs. This would make an ideal engineering school, and probably the nearest approach to this was the apprenticeship system of seventy-five years ago, where, provided the instructor was faithful and earned his fee, the final product made better engineers than the average of those turned out by the technical schools of to-day. Mr. Rafter.

EDGAR MARBURG, M. AM. SOC. C. E.—The subject is so comprehensive that an exhaustive discussion of any single phase of it would consume much time. The speaker, therefore, proposes to refer briefly, and somewhat informally, to the vital element in engineering education—the teacher. Mr. Marburg.

The development of engineering schools within recent years, though sufficiently remarkable, may be regarded, after all, as only the forerunner of an educational movement, the full magnitude of which no one, perhaps, can adequately foresee. The attendance at engineering schools has been increasing by leaps and bounds; and through State aid or private beneficence, many of the schools have been put in possession of excellent facilities. There remains, however, a certain important element of weakness, namely, the teaching element. It must be apparent to every careful observer that the teaching branch of the profession needs strengthening. Though we either have, or are rapidly getting, the ships and guns, in the way of buildings and equipment, we need more men and better men behind the guns—men of proven ability of the highest order. The field of engineering teaching has not proven attractive enough in the United States to enlist a sufficient number of men of the highest caliber. Nor is it surprising, therefore, that the teachers of engineering, broadly speaking, do not hold the same high position in the esteem of the profession as do their colleagues in other professional schools. The large influx of students into the technical schools within recent years has served to make the problem a more serious one, and its solution all the more pressing.

The need of more high-grade teachers in the engineering schools is indeed so apparent, the logic of the situation so obvious, that the speaker believes it to be only a question of time—and, he hopes that time will be comparatively short—when this need, too, will be adequately supplied. In any movement to that end, however, the loyal, active and united support of the profession, exerted through its leading organizations, would prove not only helpful, but well-nigh indispensable. The time is opportune, in the speaker's judgment,

Mr. Marburg. for this Society to co-operate in this matter by authorizing the appointment of a special committee to consider and report on the present status and needs of engineering education in the United States. The speaker proposes, therefore, to introduce a resolution to that end at the proper time.

Mr. Williams. GARDNER S. WILLIAMS, M. AM. SOC. C. E.—This problem of the proper education for the engineer is one, it is needless to say, which is very prominently before the speaker at the present time, and his doctrine is this: First of all, the civil engineer must understand the English language, and the prime essential of his training is that he shall acquire that understanding by the best and most expeditious means that can be discovered.

It is quite possible that for some the best path may be along one road, and for some it may be along another, but English is the foundation, whatever the ultimate requirements may be. Beyond that it must be remembered that to-day the engineer is becoming recognized as belonging, not only to the learned professions, but to the most learned profession, and he should be a leader of thought in the community in which he resides. In order that he may be such a leader, he should be familiar with the history of his country and the history of civilization. History, therefore, should play an important part in the preparatory education of the engineer. He should also understand the principles of common law, as well as those of the law of contracts, the functions of public corporations and of governmental agencies. He should also be thoroughly versed in physics and mechanics, and the speaker would emphasize the statement which has already been made, that what is needed is not the learning of the textbook in detail, but the learning of it in its application to the world at large—the power to apply the things which have been learned. In other words, the essential part of the engineer's training in college, if he goes into the technical school, should be in design. He should be taught to apply the principles which he has learned in the classroom; and, to the speaker's mind, those schools which are devoting the most attention to the designing of engineering work are the ones which are best fitting their students for the demands that will be made upon them five or ten years after they are out of college. Not that they should learn the design of details, but that they should learn the principles upon which all correct design is based.

There has been too much tendency of late—it is, fortunately, not so marked at present as it was four or five years ago—to develop manual training at the expense of other education, in short, to produce skilled mechanics, and many schools are devoting much attention to shop work with the idea of producing men who can go into a shop and receive perhaps as much as $4 or $5 per day upon

graduation; and who have the prospect of getting that for the rest of their lives. The speaker does not understand that it is the place of the university to fit men for that particular occupation. It seems to him that, if the student is going into the shop, all he should attempt to do—all he has time to accomplish in a college course—is to learn in general what is to be done, and what can be done. It is not necessary that he should be able to do it himself; it is not worth while to spend the time necessary to learn that. That time can be more profitably utilized in other ways. Mr. Williams.

There was a period, not long ago, when practitioners demanded that young engineers coming out of college should be able to do something immediately. Our railroad friends said, "Your college men are no good: they do not know as much as the man who works up from the shop or the chain." They do not, as far as to-day or to-morrow is concerned, but the time is rapidly approaching when their value will be realized more fully by the railroad men than it is at present. A young man can get a world of experience in the five or six years of his life after he has come out of college, and he will get it, or he will drop out of the profession; but, in his strenuous efforts for a place in the world during those five or six years, he will not get that training in the essentials of Engineering which the college education should give.

If a student has absorbed his college training, has put it away where it will not go from him, although for the first three or four years out of college he may believe that he has wasted much valuable time, though he may talk of his professors as fossils or impracticable men, nevertheless, 5, 6 or 10 years afterward, he will come to that position in his profession (after he has acquired that training and experience which all must have in order to succeed), when he will begin to see the usefulness of those courses which he regarded as drudgery; then the heads of those who have had brains enough to retain that instruction will be seen rising above those of the rank and file of their fellows.

The duty of the teacher of Engineering is to prepare the student for the position he must fill, not to-day nor to-morrow, not next week nor next month, but 10, or 15, or 20 years from now, when the engineering students of to-day must be the leaders in the Engineering profession.

Olin H. Landreth, M. Am. Soc. C. E.—Those members of the Society who attended the Engineering Congress at Chicago in 1893 will doubtless recall that the Committee of Arrangements divided that Congress into several sections, one of which was on engineering education, and that of all the six or seven sections which met simultaneously during the week none was so generally well attended, and none had its interest so well sustained. It will also Mr. Landreth.

Mr. Landreth. be remembered that, as the outcome of those sessions, there came to be felt so strongly the need of more time and attention to the subject of engineering education than this Society was able to give at its annual meetings and conventions, that a new society, the Society for the Promotion of Engineering Education, was then established and has become one of the most important engineering and educational organizations, and one which occupies an exceedingly important and influential relation to the entire engineering profession. As a charter member of that society, it is a matter of gratification to the speaker that the American Society of Civil Engineers has again thought the subject of engineering education sufficiently important to devote a considerable portion of one of its sessions at this Convention to the discussion of two phases only of the general subject.

The two important and closely related topics covered by the title have been so well presented by Professor Swain that it is not the speaker's intention to do more than refer to one or two points.

1.—The first point to which attention is asked deals with the relative value of the "successive" as compared with the "simultaneous" method of engineering education; that is, In the education of an engineer, is it better for the young man to complete his general education before taking up his technical training? It is not necessary to state that the engineer of the present and of the future needs, not only intensified technical training, but also broad, general, cultural training. No debate on this proposition is assumed to be necessary. The question here offered for consideration is: Should the young engineer complete his general training before beginning his professional studies, or should the two be carried on simultaneously? The remark is frequently heard that a young man intending to follow engineering should first complete a general college course and then enter the professional engineering school. Some effort has been made in a number of cases to ascertain just why the successive order for taking the two kinds of training has been stipulated, and in every instance examined it has been found to have originated, not from a critical comparison with the simultaneous method, but in the general assumption that there is but one way to acquire both kinds of training, and that is to take them in tandem, or, to use an electrical comparison, to take them in series rather than in parallel. No doubt the general custom followed by students of theology, law and medicine has been largely responsible for the general impression that the same plan should be followed in engineering. Another and a valid reason has existed, but is fast disappearing. In the past the college and the professional engineering school have not generally been component parts of the same institution, and thus have not been in close proximity, but separated. This has made the successive

method necessary. With the modern university, however, comprising academic and professional schools closely contiguous, the foregoing reason does not exist. Both the successive and the simultaneous methods, therefore, are now generally possible, as far as the contiguity of the college and the professional school is concerned. Mr. Landreth.

The advantage to an engineering student of acquiring his general college course and his technical training at the same time will now be considered. The profession of engineering demands of those who follow it successfully peculiar and unusual qualifications. Integrity of character, a capacity and inclination for hard work, power to overcome difficulties, to meet emergencies and to initiate and execute along definite, preconceived lines, and a mind trained to critical, analytical and constructive processes, are certainly not the least of the engineer's qualifications. To develop these qualifications, and especially the analytical and constructive faculties, under the best conditions, requires that the candidate be placed as early as possible under influences and in an atmosphere favorable to this development. It is no criticism of academic training that the above conditions exist to a higher degree in the technical school than in the general college; and it is desirable, therefore, that the student should enter the technical school as early as possible, and thus, while prosecuting his college course, in parallel, secure the stimulus and inspiration furnished by the definiteness of purpose of the technical course and its atmosphere of exact, critical and constructive ideas. When a student completes his college course before entering on his technical training, which latter may be presumed to require two years, his habits of thought have been largely formed, and his mind does not adapt itself as effectively to the habits and methods of exactness and definiteness of objective which are so essential to the successful engineer. Moreover, several of the leading lines of subjects which the technical course must contain require for their best treatment, not only a proper sequence, but also adequate time for proper mental assimilation. The educational principle of unconscious cerebration is here important. The mind requires time for the fullest assimilation of the ideas of an abstruse subject. To compress the numerous and intimately related technical subjects into two years' time cannot but interfere with the proper sequence of topics and also merges so closely on the evil of cramming as to impair seriously the educational value of the course.

An extended and somewhat careful study of the problem, covering an experience of several years, both on the side of the teacher and of the practitioner, leads the speaker to the conclusion that the following programme comprises in general the best preparatory education for the civil engineering profession for the student who is able to take a full college course in addition to his technical training:

Mr. Landreth. *First.*—Graduation from a high school or a school of equal grade, which will fit a man for entrance to college and also to the better grade of engineering school.

Second.—If a student can secure the opportunity or can devote the time, he should then spend one year, in a subordinate capacity, in connection with some extensive engineering enterprise or undertaking, preferably where a variety of work is being carried on. How subordinate or poorly paid the position matters less than that it be one from which the young man will have an opportunity to observe and comprehend the workings of the enterprise, and one where he will be able to acquire a feeling of responsibility for the performance of his duties, and to feel that he was a part, if even a very small part, of the enterprise.

Third.—The careful selection of a college and a technical school which not only furnish superior courses of their respective kinds, but which are so situated, either under the same or under different organizations, that the student may enter both simultaneously and arrange his course, extending over six years, so that the two lines of training may be judiciously and systematically laid out and carried along together. By this plan the desired sequence of subjects may be secured, and the necessary time for mental assimilation of the more abstruse subjects obtained, and both sides of the man developed simultaneously. It is not to be supposed that the time proportionally allotted to each of the two lines of training would be the same throughout the entire course. During the early years the general training would predominate, with enough of the technical, however, to lay an early foundation for technical thought and appreciation and to extend sufficiently in time the several topical lines of technical subjects. In the latter years of the course the technical subjects would necessarily predominate, and the student would come to them with the all-round development and the breadth of view which the general training had given him.

The important question as to what each of the two courses should comprise is too extensive for discussion at this time. In the speaker's opinion, however, the college course which is best suited to the needs of engineering students is the old-line classical course, with Greek omitted and its place taken by increased work in history, economics, sociology and æsthetics. For the engineering course, the increasing importance of better and broader training in the fundamental technical studies leaves little legitimate opportunity for specialization in the two years properly assignable to the technical portion of the course. A thorough mastery of the fundamental principles of those subjects which should be familiar to every engineer, whatever his specialty, leaves no time in the course for narrow specialization; that should come

later. Technical specialization in an engineering course undoubtedly leads to more immediate utilitarian results, but invariably at the sacrifice of breadth of training and future efficiency. The profession appears to be already over-supplied with the specialist whose narrowness of training and whose consequent unfamiliarity with the fundamental principles of the neighboring branches of his own profession, and frequently with life itself, impair his judgment and limit his usefulness to society. It is bad designing which increases the altitude of a structure by materials needed in the foundation. The Eiffel Tower, with its broad base, symmetrical and graceful outline and superior altitude, furnishes an excellent model in simile for the training of an engineer. Mr. Landreth.

2.—Turning to the second question of this topical discussion, *viz.*: "Is Technical Training the Best Education for Executive Work?" it would seem from Professor Swain's treatment of the question that by the term, executive work, he had in mind administrative management. With this interpretation it would seem that the answer must depend very largely on what kind of administrative work is contemplated. It is possible to mention some lines of administrative work in which a technical training would not be of especial value and might even be harmful, if it had absorbed time otherwise available for a training more pertinent and directly related to the administrative work in question. There are, however, a great number of lines of administrative activity, and, under modern conditions and tendencies, the number is rapidly increasing, in which a technical engineering training forms an important if not an essential part of the preparation. In fact, engineering is coming to comprise more of administrative work than ever before. The work of the modern engineer may be classified or analyzed into the following divisions:

Investigation,
Design,
Execution, and
Administration.

Before beginning any enterprise or undertaking the engineer investigates conditions and circumstances to determine the possibilities and limitations surrounding the undertaking. With the result of this investigation before him, if favorable, he designs his enterprise, his structures, his methods and accessories. He then executes the undertaking from the designs; and, if the enterprise be one involving technical skill in its management, he or his brother engineer proceeds to administer it. It is only recently that the fourth duty, that of administration, has fallen to the engineer; formerly the business man took this part together with a

Mr. Landreth. majority of the credit and emoluments of the undertaking. Since technical skill has now become an essential adjunct of the administration of many classes of industrial and even business enterprises these questions have arisen:

a.—Shall the administration be divided between a business man and an engineer, or shall there be but one administrator?

b.—If but one, shall the business man be given an engineering training, or shall the engineer be given a business training?

The principle, accredited to Napoleon, that one poor leader is better than two good ones, in recognizing the necessity of unity of responsibility, may be accepted as representing one of the important principles of modern administrative systems, *viz.*, the principle of undivided responsibility.

If we are to have but one administrator possessed of both business and technical skill, economy clearly dictates that it is better to give the engineer the business training than to attempt to give the business man the technical training.

If the engineer is to become an administrator, the question at once presents itself: How can he best acquire his administrative ability? Is it best that he should acquire it entirely through experience, without any direct aid from the schools; or is it possible to include, in his technical course of study, instruction which shall directly aid him in developing the administrative capacity after beginning his professional work?

After a careful study of the foregoing question covering several years, including observations from experience, the speaker is strongly inclined to believe that it is possible to include in the student's course of study, instruction and training which will help to lay the foundation for the development of administrative capacity after leaving school. No one of authority will claim that the engineering school makes engineers, nor that the law school makes lawyers, and yet, to-day, few will deny that the technical training is of very material assistance in making engineers as well as other professional men. What is true of the course of study at large is also true of its component parts; a study of the principles directly underlying drafting, surveying, mechanics and materials substantially assists a man to become a draftsman, a surveyor or a structural designer; and it is also believed that proper instruction in the principles of proper subjects during the student's course of study may also be of direct value toward giving the engineer ability in the administrative direction later. Of course, nothing more than fundamental principles should be undertaken, but these should be well taught. Any attempt to give bare information, methods or procedure, beyond what is useful in explaining and making

clear the principles, would doubtless result, as in most other technical subjects, in loss of time and disappointment. Mr. Landreth.

In addition to the regular courses in history, economics, sociology and political science, which the engineer's general college course should include, the particular subjects which appear to promise the most useful results, as further aids in laying a foundation for administrative development, would appear to be:

(1).—The elements of law in general and of the law of contracts, agency, property, bailments and corporations, in somewhat further extension;

(2).—The principles of finance and financial operations, with applications to the monetary, credit and exchange systems in vogue;

(3).—The principles of accounting in general, and further as applied to transfer accounts, stock accounts and cost-keeping systems;

(4).—The principles of business administration, more especially as applicable to organization, dicipline and effective execution.

With a framework of principles such as the foregoing or other equivalent subjects, firmly established, the young engineer on going out into professional work would have a substantial foundation on which to build, and would understand and absorb much more effectively and rapidly the facts and conditions of his experience and would assimilate them more readily in the development of his administrative capacity.

Bernard R. Green, M. Am. Soc. C. E.—Although the proposer of the topic for this occasion, the speaker's object was not to make an opportunity to offer ideas, but to secure an answer to the question or to arouse discussion which might lead to one. Professor Swain's views are very near to those of the speaker's, the result of a lifetime of observation over the period—so well known to the grandfather engineers of to-day—which has covered perhaps 90% of the stage of development of the science and art of civil engineering. Mr. Green.

The question of education for the engineering profession must surely be a very present one to the minds of, at least, all the older engineers. The speaker's experience began when preparatory schools were few and weak. Other than the Rensselaer Polytechnic Institute, the United States Military Academy, and two or three new, so-called scientific schools attached to classical colleges, there was none. The word "attached" is used because these schools had so little in common with the college course proper that they might as well have been located anywhere else. The speaker took the civil engineering course at the Lawrence Scientific School, of Harvard College, in 1861, then a very small school in almost every sense, but hardly inferior to the Rensselaer Polytechnic Institute, if the

Mr. Green. student were uncommonly well prepared and industrious, for he could not graduate without passing a searching examination. Consequently, though the classes were small in numbers, hardly one-third of their members were graduated. The course consisted chiefly of drawing, the higher mathematics, and the mathematics of surveying and mechanics, with no laboratory work, except a few lessons in the use of surveying instruments, and no visits to engineering works finished or in progress. Two years of this work brought the class to graduation with the degree of S. B. Those students who had previously taken a part or the whole of a college course usually had the advantage of the others.

Entering upon work in an engineering office the speaker began to read engineering books and to review the textbooks of the school. Observed by the Chief to be thus engaged, he was encouraged to continue because, as he remarked, by reading something about engineering would be learned, but one could not thereby alone become an engineer. Due recovery from the jolt of this surprising statement followed when the speaker had had time to appreciate that no graduate of a school could possibly become a real engineer without practical experience and observation.

Since then, through life, he has endeavored to be a civil engineer, while the professional schools have developed and multiplied, and the science and art of engineering have grown to the enormous proportions of the present day; it is an amazing contemplation, whether measured by quantity, quality and territory conquered, or by scientific development, refinement and the sum of knowledge gained. This period of forty-five years has comprehended almost the whole growth from nothing to much, and made civil engineering, considered in its broad sense, a profession coequal with the highest and most dignified as well as the most indispensable to mankind. All this is naturally best realized by those engineers who have been the authors of the development, many of whom have at times regretted that they had been born so early, and have envied the advantages of younger men; but, the older men, at least, realize that the demands upon the profession have, nevertheless, outrun the schools, and that there is great need of a higher and more thorough and comprehensive character of school training than has yet been provided.

Such engineering institutions of learning are sure to come in some way, but the day should be hastened with all safe speed. The universities are endeavoring to solve the problem, but they are hampered for means of all kinds, financial, physical and pedagogic. The recent effort to combine the Massachusetts Institute of Technology with Harvard University is most significant of what is needed for the best education of an engineer. There is much profit-

able thinking and discussion possible in all this matter, and immeasurably good results to be gained, as evidenced by the active existence of the Society for the Promotion of Engineering Education. Mr. Green.

Perhaps this subject is more interesting to the speaker than to engineers who have not been called upon to steer one or two sons into the profession. In due course the speaker found himself with a boy growing up, who had a decided bent for engineering, and he was given the civil engineering course at the University of Michigan. He has since been in constant practice for sixteen years. It was the best that could be done for him at that time. Now another son, with a similar bee in his bonnet, is taking the school course, but he is endeavoring to widen and perfect it by a combined college course, which until recent years was rarely done. Even now this course is too uncommon, but the universities are not economically organized for it, and few boys can afford the necessary time and expense. In the present instance the young man is so choosing the electives at Harvard University that he may secure the A. B. degree and then finish the civil engineering course at the Massachusetts Institute of Technology in two more years with a C. E. degree, a course covering six years in all. Thus he may secure the advantages of a so-called college course, with the humanities—English especially, and plenty of it—together with a pretty thorough curriculum in civil engineering proper. His aim is ultimately to become truly learned in his profession, in a broad way, and to be able to hold his own in contact with educated men and gentlemen anywhere.

This should be the training and accomplishment of every engineer. Few others can rise to the top of the profession at the present day, and the best engineering schools must be so organized, connected with the higher education in general, and equipped with a physical outfit and a manly and cultivated personnel in their teaching corps, that they may ensure to the students such training. Think of some of the professors and teachers to be found in many of the best schools of the day—kings in mathematics, wizards in chemistry, magazines of engineering facts, but no more magnetic, graceful, or sympathetic than an automaton. The professors and instructors should be sound, enthusiastic and practical men of experience, as well as scholars in the technology of the profession, and attractive gentlemen withal. What might not an ambitious and enthusiastic young student of the grand art of engineering accomplish, in his brief course at school, under the guidance of and by coming in personal contact with such teachers! How well and with what gratitude do many of us remember some one or two, of all those under whom we worked at school, as our ideals of ex-

Mr. Green. cellence! But salaries are small—very small—and many of the teachers, and too many of the professors themselves, have had but little practical experience. The situation will be different when salaries tempt and universities insist on practical engineers with the teaching faculty and all the other qualifications necessary to those extremely important positions. The speaker suffered much in his school from lack of excellent teachers—although Professor Eustis, as the head of it, was ideal—but the members of the class were quite sure that one of his assistants learned much more about mathematics from them than they did from him.

The engineer of the present day occupies an enormously important and responsible place in the world which is constantly demanding more and more of him at his best. Engineering undertakings are increasing in number, magnitude and complexity. At first, engaged in the ingenious application of skill and the knowledge of materials and physical forces, in a few isolated places and in a small way; then in a wider extension of this skill and of mechanical development; and at last in the application of still higher principles of engineering science, in the world-wide relations of engineering to almost all interests of mankind, this has been the scope of the profession of the civil engineer.

How should the boy and the student engineer be trained by the university in this day and generation? That is the question, and it needs a much more definite answer than has yet been presented. Each father, especially if he be an engineer himself, is endeavoring to determine this for his son by selecting from the various schools that course, both mental and manual, which may accomplish the best results for his own particular boy in the few years—four to six—which can be afforded for it, to the end that he shall acquire a good knowledge of history, literature and language, of how to think and study and investigate, of how to speak and write, and a habit of general reading, as well as a knowledge of the technics of the profession.

Students to attain this end must come from good family influences, and be healthy, interested and assiduous. The engineer should be refined, brave, self-reliant, ready with clear-cut ideas and good English in speaking and writing, and thus be able to make an effective appearance, and to give a good account of himself before any board, committee, or individual, at any moment. Such men command respect, a hearing, and then, very often, the whole situation.

The young civil engineer should strive to learn and to become familiar with mechanical operations of all kinds and with their approximate costs, matters that may be learned by observation during the school period and immediately afterward. He should

particularly learn how to judge of and to treat and handle men properly. In short, he cannot possibly learn too much, or become too skilled in anything, the physical sciences, law, literature, history, modern languages, mathematics, the technics and practice of the numerous branches of engineering, and, especially, in the fine arts, that he may never walk nor work without a present sense of the beautiful—of the graces and harmonies of things. In his student days he need never fear that any item of knowledge or skill which he may acquire, from milking a cow to analyzing a bridge truss, or singing a solo, will ever come amiss, whether it seems to pertain to his profession or not. Mr. Green.

The engineer's business is world wide, comprehending an infinite variety of applications, places, peoples and human interests. He may have to go anywhere at a minute's notice, travel widely and find himself in contact with numerous conditions calling for all the knowledge, tact and practical skill that he may possess.

Engineering literature, descriptive or critical, of undertakings or accomplished works, or of the experiences of difficult construction, is thin and scarce. Engineers have not written enough, do not write well enough, and do not like to write because they do not know how. This is because of the insufficient training of the past generation, which prevented ease and fluency in writing, by one intensely engaged in practical work. Consequently, many valuable experiences and ingenious accomplishments, which might have been invaluable to others and to the world, have lived and died in the minds and memories of the engineer geniuses, to whom the labor of concise and readable records was irksome and laborious.

The engineer who can stand before a lay committee of business men and explain his plan attractively and concisely, showing thorough familiarity with the subject—for hard-headed business men are pretty acute judges—is more likely to get the job than one who, though knowing it thoroughly, presents it badly.

This general question of engineering education has been brought more forcibly to the speaker's mind by the recent organization of the Washington College of Engineering under the George Washington University. Its location and prospects are most favorable, and the college already has some 200 students. In its interest, as a trustee, the speaker recently accompanied a committee of inquiry and inspection to five of the leading engineering schools of the Eastern States. All were found to be more or less overcrowded with students in the laboratories and drafting-rooms, and to need more space, more teachers and more of everything requisite to good work. Some schools appear to overdo in laboratory work, to the detriment of fundamental principles, and others to err in the opposite direction. The necessity of a consensus of practice was notice-

Mr. Green. able, such as may possibly result from the deliberations of the Society for the Promotion of Engineering Education, recently organized.

Engineering, in the general sense, now occupies more than half of the whole field of education for professional life, and all the schools devoted to it are filled, even to crowding, with students. It is easy to see the condition and trend of the times. The age of applied science has but begun. Research, keen competition in ingenuity and the mechanical performance of the world's work, the growing appreciation of public art and its recognition in engineering works, are rapidly absorbing the attention of the whole world, and it is all the engineer's business. He is involved in it everywhere, and the character and quality of the work, and its general effect for the good of the community and of the world at large, will depend on the character and quality of the engineer. As heretofore, but more than ever at the present time, the material, and, in a great degree, even the art world, depends on the engineer, and cannot get on without him.

How indispensable, therefore, his proper training and education! With what care and consideration should he not be selected, trained and launched into his work! He spends the people's money, and, in the construction of permanent and necessarily costly works, transforms topography and determines largely the permanent taxes and the cost of the future maintenance of the property. He cannot know too much, nor be too well-grounded in his profession.

Therefore is it important to consider and determine, as engineers are accustomed to do, what is the best course, and how much preparatory or school education is necessary for the professional civil engineer, and how it may be most effectively and economically obtained.

The speaker thinks it is a subject of immediate and proper concern for this Society, which should take it up for active consideration and determination in co-operation with professional educators at large.

Mr. Wendt. Edwin F. Wendt, M. Am. Soc. C. E.—In discussing the question, "What is the Best Preparatory Education for the Civil Engineering Profession?" a previous speaker commented on the fact that the American college or university does not seem to aim to educate young men to be gentlemen. This remark suggests that Ruskin at one time made the statement that the object of the English university was to turn out both scholars and gentlemen. The American university, apparently, makes no direct effort to turn out more than scholars, but it should be the object of every American school of civil engineering to graduate young men who are gentlemen as well as scholars.

Mr. Wendt.

The education of the young man who expects to be a civil engineer should be most thorough and comprehensive. A good common school education is essential. The college course which gives the greatest and most thorough training is probably that which leads to the degree of A. B. Having received this degree, the student of civil engineering should enter a technical college and complete the special course which is related to his future profession. After the completion of the courses of the common American college and the technical institution the young civil engineer should be placed in an office where he may become acquainted with the methods pertaining to the design and execution of engineering work. A student can afford to accept such a position, even without compensation, in order to become acquainted with the routine of work. Without this knowledge, a graduate is unable to apply his education to a useful purpose. While the student is getting all this preliminary education, he will find it greatly to his advantage to obtain during the summer vacation a position on an engineering corps, which will bring him in contact with field work, in order that he may become acquainted with the routine of the field as well as the office. Such positions are hard to obtain, and the wise student will endeavor to get an opportunity of this character even if the compensation should be only sufficient to cover expenses. The ambitious student, who is willing to work at a small compensation in order to gain this valuable experience, will always be able to get the desired opportunity.

The second question under discussion: "Is Technical Education the Best Training for Executive Work?" does not seem to be well stated. It should read, "Should Technical Training Form a Part of One's Education for Executive Work?" An affirmative answer, only, can be given to this question. In former times civil engineers who became executives generally failed to get this technical training. Such schools were not numerous, and their location was at a distance, so that not many young men could bear the expense of pursuing a course in a technical institution. At the present time, however, these institutions are quite numerous, and are well located geographically, so that a technical education is within the reach of a large proportion of college men.

Competition, among civil engineers who desire executive positions, is very keen at the present time, and experience seems to indicate that the man who possesses a technical education will advance more rapidly than the man who does not. The successful management of the business of the large corporations of the present day depends upon the judgment of the executive officers, and the man who possesses technical as well as practical knowledge should possess the best judgment, and, therefore, ought to be well qualified for executive work.

Mr. Wendt. The time is not far distant when a large majority of the executive positions in the business world will be occupied by men who have had a technical education.

After all this has been said, the fact remains, that many of the men occupying executive positions in the United States have not received any technical education. They entered business life early, studied human nature, and gradually became well fitted for handling men. Experience was gained day by day in the practical school of business, and such men have been very successful. The boy who obtains a technical education should outstrip the one who is not so fortunate, but, if he does not possess a knowledge of human nature, he will become a very poor executive, no matter how thorough his education. Executives, like poets, are born, not made. Those possessing technical education as well as practical experience should be more efficient than those possessing technical education only.

Mr. Talbot. A. N. Talbot, M. Am. Soc. C. E.—Mr. Green has referred to the importance of this topic, and has expressed the desire that the Society give it further attention. Professor Landreth has mentioned the Society for the Promotion of Engineering Education, which society has become a vigorous one. Its policy for the past few years has been to hold its annual meeting in connection with some engineering society. Three years ago there was a joint session at Niagara Falls with the American Institute of Electrical Engineers, resulting in a very interesting discussion and excellent papers, from the standpoint of the engineer and of the educator, on the subject of Engineering Education. In 1905 the meeting with the American Society for Testing Materials was productive of much good. This year it will meet with the American Association for the Advancement of Science.

In the past, the Society for the Promotion of Engineering Education has not been able to make arrangements for a joint session with the American Society of Civil Engineers. It seems to the speaker that much would be gained in a discussion of the requirements and needs of Engineering Education by having a session of this Society in connection with the Society for the Promotion of Engineering Education, and it is suggested that arrangements be made for the consideration of educational topics at a meeting of this kind.

Mr. Coates. F. R. Coates, Assoc. M. Am. Soc. C. E.—In a book published by E. H. McHenry, M. Am. Soc. C. E., several years ago, the following definition is given: "Engineering is the art of making a dollar earn the most interest." In a great many instances engineers do not pay enough attention to this, and the failure of a number of men in the profession is distinctly attributable to the fact that,

instead of following Mr. McHenry's definition, they controvert the same and make it read as follows: "Engineering is the art of making a dollar go the quickest." Mr. Coates.

In order to obviate this, the young engineer should acquire, at the proper time, a commercial and financial training, as applied to his profession. He should also have instilled into him the practical side of engineering—a theorist will never be a success as an engineer. By the time he has completed his four years' course at college, he should have determined the special branch of engineering which he is going to follow; then he should connect himself with a company where he can get two years' field work, then a year in the drafting-room, followed by another year in the field. After this, in order to broaden himself, he should become associated with a contracting company for two or three years. During his connection with the latter, he will have impressed upon him, to the fullest degree, that Mr. McHenry's definition of engineering is about correct.

A man who follows this course, will find that, at the end of the time thus spent, he will be thoroughly qualified to be placed in responsible charge of work.

While on the subject of the "technical training of an engineer," one point should be emphasized, and that is in connection with engineering instruction in colleges and universities. A thorough course in maintenance engineering should be given to those students who intend to follow railroading. This department needs special care and attention, as it is one in which money can be saved or wasted very rapidly. A great deal of practical work is needed by the engineer who follows this branch, but it opens up a large field, and, at present, the practically trained engineer of maintenance is a scarcity.

Bassett Jones, Jr., Esq. (by letter).—The essential thing in education is to give the student, as Mr. Swain says, "the ability to think clearly and logically," but before this can be done there must be aroused in the student an interest in clear thinking, or, which is the same thing, in logic. Logic is the empirical science of thought, in the same sense that biology is the empirical science of organic activities, or that mechanics is the empirical science of moving masses. The writer would thus distinguish sharply between philosophy and the empirical sciences of so-called given facts, which, in reality, go no deeper than the facts themselves, and which, as such, teach nothing of the actual internal structure of the world. Such sciences are the sciences of "the world of description," and, as this term implies, merely serve to translate into terms of thought what experience of this world finds to be facts. They are adequate only as a means of unifying the common experience of many individuals. Mr. Jones.

Mr. Jones. That this is so, is easily shown, for who among men has really experienced, for instance, the law of gravitation? Who can say that such a law really exists? Of course, it is only an approximate description of one small fragment of our social experiences, and serves only to describe in what order these experiences may be expected to take place. The law itself is valid only as far as our experience now goes, and never states any deep truth as to the nature of facts, the behavior of which it may thus be said to describe.

It is easy to say that such considerations as these are not "practical." Engineering does not attempt to go behind the scenes. It is satisfied to do what it can in dealing with the world as science finds it, and in putting into concrete shape those laws which the abstract scientist states as adequately describing, for practical purposes, the various activities of Nature. But here is the point. What are these practical purposes? Are they no more than a blind groping after something unknown? How can we grope for something of which we know nothing? Is life after all such a game of pure chance that we may be likened to blind men seeking for the light? If we know nothing of that for which we search, then indeed we could not even know that it existed, and then, pray, whence the search? But if we do know that, somehow, and sometime, the purposes of this life are fulfilled, and if, therefore, we know that this fulfillment has real being, then let us pause to ask ourselves how we know so much, and whence comes our knowledge. Such an inquiry must give us a standard by which to gauge our purposes, and thus only can we learn how we ought to live in order that our life may be given some real value.

Does it not appear on the face of it, that what an education, at least, ought to do for us is to teach us how to estimate and judge life? If the student knows nothing of the chief business that is to engage his efforts, namely the business of living, then how is he, first of all, to select for himself a proper field for his endeavors, and how, when he has found his opportunities, is he to make the best of them? Does the engineer lay the cornerstone before every detail of the structure has been worked out? Then, how wise is it for a man to begin life not knowing what he ought to do next, and without any definite plan and purpose in hand.

It will be claimed, and with some justice, that it is just here that professional education has the advantage over classical training, inasmuch as it gives the student a definite plan in life—a means of gaining his livelihood, and an interest which should color the world for him with a meaning which it could not otherwise have. But does it not appear at once that the world is not the field for any special training alone? The world is not all manifestations of the validity of electrical theory, any more than it is the field in

which the theories of statics alone hold sway. It is the manifesta- Mr. Jones.
tion of conscious life, and as such it should be studied. That for an individual the laws of thermo-dynamics possess a particular interest is no reason that, for him, the world should be only a huge case of the applicability of certain differential equations. The writer has been with men who were dead to facts, and, as far as he could make out, dead to themselves, unless they stood in the presence of certain types of machinery. Such men are not intelligent beings, they are neither selves nor individuals. It seems to the writer that rather than be like that, it would almost be better not to be at all.

It is easy enough to say that no knowledge but useful knowledge should be taught, but that depends on what is meant by "useful." A man's life may be so narrow that very little knowledge will be useful to him outside of one or two subjects, and, again, a man's life may be so broad that he will find some application for nearly all knowledge. In fact, knowledge is never the criterion; it is always life. Then the question naturally arises, for what is any knowledge useful? Is it useful only as a means of gaining a livelihood—"to fill the till and the larder?" Then are we no better than the beasts of the field? Is knowledge only of value as a foundation for constructive achievement? Alas, all this we must leave behind us, unless, indeed, it is torn down and superseded ere we go. Yes, you say, but each thing done is something added, and lives again in what comes after. But if the small fragment of my purposes which I leave behind me embodied on this earth is all that is to give real significance to my being, then indeed how wasted is my life and struggle for self expression, for do not the heavens show us how relatively short-lived is even this little planet of ours?

The acquiring of knowledge has never been able to halt at the point where knowledge ceases to be purely utilitarian.

"Natural knowledge in desiring to ascertain the laws of comfort has been driven to ascertain the laws of conduct, and so to lay the foundation for morality."*

All knowledge is useful if rational, that is, if the facts are in harmony with the deepest purposes expressed in the world. "A thing is useful," says Plato, "if it is sought after with a view to the beautiful and the good; but if pursued in any other spirit, useless." "If facts are irrational, and therefore of no value in establishing principles of conduct, which is the chief use of facts," they are useless. The facts of my life are to teach me how to live, and the richer my life, the larger the body of organized facts that are needed to make up my knowledge. But the word "organized" must be used with circumspection, for there is no fact discoverable in

* Huxley, "Advisableness of Improving Natural Knowledge."

Mr. Jones. this world that bears no relation to any other fact whatsoever. No fact is utterly independent of the rest of the world. Such an independent fact would be a complete anomaly. When we speak of organized facts we mean those facts which are the organized part of a given life purpose. Thus, if it is my purpose in life to be a botanist, the discoverable facts of plant growth, behavior, and variations are thereby found by me and organized in my attempts to interpret consciously their meaning. If this purpose, namely, to be a botanist, were my sole purpose in life, then indeed would all other facts, in so far as they bore no definite relations to the study of plants, fade into insignificance, and would be classed by me as "useless." But it is necessary here to note that a life devoted so exclusively to one class of facts would be utterly impossible, for no one aspect of our complex world could be thus separated and treated, without consideration for the remainder. Such simple-purposed lives are found only in the lowest animals, where, for instance, the obtaining of food seems to be the only actuating principle.

We are limited, however, by our own stage of finite consciousness, to an attention which can only grasp relatively little of the world. This is because we fail to find more than a relatively narrow purpose in life, but, however barren our existence may seem, better and more complete purposes are always evolving with the progress of conscious life toward a fulfillment which, of course, we can never hope to find in this temporal world or in this stage of being. But that this fulfillment is, we have good warrant for believing.

How is it, then, that we may hope to approach still nearer to our ideal—the ideal of a life that finds its purposes completely fulfilled and all conscious experience organized into one perfect whole? Our only course lies in observing what the progress of the world appears to be, and in seeking to develop our lives in harmony with this law of progress. A life that is, thus, in accordance with what we may, perhaps, call the World-Purpose, is a truly moral life, and will thus express the best form of conscious being.

Our only means of discovering in what direction this tendency lies is by a process of trial and error. And this we use whenever we tentatively try a plan of procedure, whenever we experiment with facts and observe their behavior. This process is the role of pure science. If the results of our experiment harmonize with our already organized experience, we accept the modification and apply the new data to our life, thus producing a corresponding change in our own behavior. This new scheme of conduct we apply again and again to each new episode as it arises, until we find that further revision of our code is, for our individual life, necessary. Thus every process tends to become recurrent and to develop a perma-

nent "set" in the life of the individual. This process of development we find particularly evident in the life of the young conscious being before he has found a plan in life and before he has learned what he ought to do in order to win as much fulfillment of his purpose as it deserves. Mr. Jones.

Now the growth of a self is due purely to its social environment. An adequate model must be provided for purposes of imitation. The deeds of this model the young self will copy, but where it finds that such deeds are not applicable to the development of its own individuality it will interpolate new deeds that may or may not meet the exigencies of the occasion. In this manner a method of procedure will be found that will give some form of expression to the new life. But every such intermediate deed will tend to become recurrent if the result is found to be acceptable, in accordance with what the self hopes to do. Some are satisfied with less, and some require more; so that it becomes quite evident that, if we are to develop high ideals in the young individual, we must give him the widest knowledge possible of what under a great variety of conditions have been the deeds of his fellow men. This he should have before he is allowed to select for himself any particular field of work.

The fundamental defect in our present methods of technical education seems to consist in this, that the tentative period of discovery is entirely ignored. The student is never given breadth of new experience. His life plan becomes recurrent at the outset; and here the writer disagrees with Mr. Swain most heartily; he does not believe that the student should be allowed to select for himself a profession until after a long period of broadening studies. He should not be permitted to look on his education as a mere fount of knowledge which he is to apply to his selected experience of the world. He should be compelled to approach every problem as an experiment, a new source of experience, toward which he is to behave in such a way as his judgment dictates. If his judgment is defective, then his teacher is to aid him in suggesting new processes of conduct, but always leaving to the student the final judgment as to what is right and what is wrong. His model should never be such that he can imitate it deed for deed, but should only suggest to him a course of action between the deeds of the model and his own conscious activities. The importance of thus keeping the young mind in the first or experimental stage as long as possible must be palpably evident. Each new mode of conduct thus developed is a distinct addition to the ability of the student to adjust himself adequately to the fixed environment of his life. His recurrent stage of being, his permanent mode of behavior toward this environment, should be developed not in response to the demands of

Mr. Jones. the artificial world of his school, but in response to the demands of the actual world in which he is to find his place. This he can find only in so-called practical experience.

Engineering is pretty much the same in any field. It differs only in the sub-classes of facts discriminated. And for this very reason the student should be given a good foundation of general scientific studies, with other selective studies thrown in to add a little zest to his life. After this preparation, which the writer grants, should be begun early in life, the student is able to "choose a profession" with justice both to himself and to the world. This will take time? Of course it will take time—the longer the better, both for the future engineer and for the public, and, of course, the writer is speaking of the majority of engineers.

Of course, it will be readily seen that the engineer is simply an individual who has found his plan in life, and is now applying, recurrently to his experience, the processes he has learned during the more tentative, receptive and progressive period of his being—progressive, because it is very generally during this period alone that opportunity offers itself for original addition to our interpretation of Nature. No man ever wins a purely recurrent form of behavior, because the human consciousness is never able to grasp more than a small fragment of the world at one time. A type of being whose knowledge is so deep that experience comes to it only as facts to which a further application of its present plan of conduct is possible is a super-human self. Some men never attain to a fixed scheme in life, and it is among such men that the idiot is usually to be found. Other men, who have found what kind of activities can best give expression to their life purpose, still retain the power of originating new deeds to a very remarkable degree. To such a class belongs the genius, and in this class belong all those men whose lives have exerted the greatest elevating influence upon the world.

The only measure of the worth of knowledge, then, is whether it broadens and develops the "self." Whether the expression of the "self" requires more or less knowledge for its development is the only criterion of success. The successful life is the life of the man who needs much of the world to express his life purpose, for indeed, the world is no more nor less than the most complete expression that the "self" can gain in the short span of human life.

The fundamental object of education should be to arouse the student to a perception that it is not the world that limits his life, but that it is his life that limits the world. He should be shown first what relations he and the world bear to one another, and not be urged to swallow as much of the world as he can. Let him, if he can, appreciate the fact that science is never all truth, that the

world is not a huge physical laboratory, but that it is shown to us only in terms of conscious purposes seeking their fulfillment, and that the broader, deeper and nobler his purposes, the truer the world will be for him. He may then rest content, if he finds that, in his life, even a small fragment of his chosen work can be completed. He has at least learned what it is to be, and he will know that, however much of his life has been in error, whatever is worthy will find its goal and its fulfillment. For, were there no fulfillments, there could be no purposes to be fulfilled. Mr. Jones.

All this can, and should, be taught as an integral part of the curriculum, not necessarily as a special study—although the writer is convinced that at least a course of lectures could be advantageously added—but indirectly by a modification of the usual methods of instruction due to the employment of men who are teachers, in the real sense, men who, intuitively or otherwise, know enough of the psychology of teaching and the nature of the young mind to make their pupils think, not after any haphazard manner, but logically and reflectively, because they understand and appreciate the nature of the contrast between the world and their ideas.

To the complaint that men of the right caliber say they can find better and more lucrative employment for their abilities in the so-called practical walks of life, answer can be made that an educational system that turns out men so egotistical, so selfish, so dead to the higher duties of life, that they can find it in their hearts to make so dubious an excuse is, *ipso facto,* a bad educational system.

One more word as to the longer period of studies; teachers argue that public opinion is against the six-year course. What if it is? The duty of the teacher is to educate the public up to a point where it will know what is good for it.

George F. Swain, M. Am. Soc. C. E. (by letter).—The writer has been very much interested in the discussions upon this subject, and will comment briefly upon a few of the statements which have been made. Mr. Swain.

The question raised by Mr. Bouton, with reference to the elimination of incompetency, opens up a rather large subject which it does not seem appropriate to pursue at this time, as it has only an indirect relation to engineering education. There are probably incompetent men practicing the profession of engineering, who have been graduated from technical schools, and there are very competent men practicing it, who have had no training in technical schools. The time will come, perhaps, when some guaranty of a man's ability will be required before he is allowed to practice as an engineer, but the objections to any requirement of this kind, which Mr. Bouton himself has stated and quoted, seem to the writer quite strong. It

Mr. Swain. may be remarked that practitioners who belong to the class designated by Mr. Bouton as frauds probably do not have as much chance in engineering as they have in medicine, for, in the case of the latter profession, the matter of sentiment has a very strong bearing, and people are governed largely by it when dealing with questions concerning their own health. Probably in engineering it will be found impracticable to eliminate incompetent men until the employers of engineers are sufficiently educated to be able to distinguish competency from incompetency.

The writer dissents from Mr. Rafter's plea for smaller schools. There are advantages and disadvantages in large schools, but it seems that a very essential point is the number of students per instructor. If there are as many instructors in proportion to students in a large as there are in a small school, there is a distinct gain in the former, on account of the specialization which may be introduced, so that the student meets many men instead of a few, and these men are presumably better posted in their special lines; although, as a rule, he may not come into as intimate or as continuous contact with any one man as he does in a small school. If a student by going to a small school can come into intimate personal relations with some very able and inspiring man, this contact is of great value to him, but the chances are that in a large school he will meet more and abler men, and gain more inspiration. Moreover, the stimulus of a large number of students working along different lines is of importance. The writer is very much surprised at Mr. Rafter's assertion that:

> "It is within the experience of everybody who has studied at a technical school that, during the first year or two of practical work, he is chiefly engaged in unlearning the things learned there, and during the next year or two he mostly forgets the things learned—at any rate, so far as mere memorizing is concerned."

If this statement is true to any considerable extent, it is a severe reflection upon the present methods used in our engineering schools. Undoubtedly much improvement may be made in methods of teaching. There is much mere parrot-like learning from books, and memorizing of formulas and rules, but this is not teaching, and the intending engineer should shun schools, if there are any, where such methods of study are the rule. There are many schools where the object is to secure true mental training, and where the attentive and industrious student can secure this training.

Several of those who have discussed this subject have suggested the appointment of a committee of this Society to consider the general question. The writer is heartily in favor of such action, and believes that a properly constituted committee may accomplish good

in raising the standard of engineering education, and establishing a recognition of its desirability by the profession and the public. Mr. Swain.

Professor Landreth's argument in favor of taking one continuous course of study, instead of taking first a college course and afterward a technical course, is very strong, and emphasizes the writer's remarks and opinions. Such a continuous course is practicable in a university with which a technical school is connected, and the writer feels that it is proper to call attention to the fact that it is also practicable in some independent technical schools, as, for instance, the Massachusetts Institute of Technology, which has always insisted on the desirability of including some general studies in an engineering course, and has always provided a large staff of teachers for such general studies. The last catalogue of this institution shows that in a faculty of 75, there are 7 in mathematics, 3 in biology, 2 in economics, 3 in English, 4 in geology, 2 in history, 2 in modern languages, and also a large number of instructors in language and general studies. As it becomes more fully realized that the engineer must be a broadly trained man, it will be more fully realized that our technical schools must do one of two things: they must either make their courses graduate courses, requiring a college degree, or, for admission, must have a substantial equivalent in training; or they must include in their own curricula a considerable amount of training in culture studies, increasing the length of time devoted to the course. It will be found essential that the independent engineering schools have strong faculties in general or humanistic lines of study. If so provided, these independent technical schools will be, in reality, colleges giving both a general and technical education in one continuous course, with the added advantage that comes from the fact that the students are working with an aim; that they do not go to the school primarily to have a good time; that they are more earnest, industrious and interested in their work than the average student. A course of six years, with one year of practical work preceding it, as Professor Landreth suggests, seems to the writer, however, rather longer than the average young man would be able to afford.

The writer thoroughly agrees with the emphasis that Mr. Green lays on the breadth of training which is needed for the engineer of the future. He cannot agree, however, with Mr. Jones that the student should be allowed to select a course for himself only after a long period of broadening study. His experience and judgment alike convince him of the tremendous advantage which comes from having an aim, and he believes that a young man of eighteen, with the advice of teachers and parents, should be able to choose his career within broad general lines. He can decide, for instance,

Mr Swain. whether he wishes to follow some scientific pursuit, or whether he wishes to follow law, or economics, or literature. He can thus enter upon some general course of study, not leading at first to a specialized profession, but only in a well-defined direction, and, in the course of a year or two, he will be able to narrow his aim still further, until, in the last two or three years of his course, he will be aiming directly at some one of the recognized professions. For instance, the writer believes that, during a technical course of four years, the differences between the courses of civil, mechanical and electrical engineering should be slight during the first year, or even during the first two years. As a student advances in his subjects, he will be better able to discover the course for which he seems to be fitted. No amount of browsing around upon philosophy, language, or economics will help him to do this. By studying chemistry he will decide whether he is adapted for chemistry, and, if so, he can direct his work toward chemical engineering, applied chemistry, mining engineering, or sanitary engineering; and if he finds that subject repugnant to him, and that he is interested in mathematics, descriptive geometry and machinery, he may turn toward civil, mechanical, or electrical engineering.

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TRANSACTIONS.

Paper No. 1031.

STREET TRAFFIC IN NEW YORK CITY, 1885 AND 1904.*

By Clifford Richardson, Assoc. Am. Soc. C. E.

With Discussion by Messrs. S. Whinery, N. P. Lewis, G. W. Tillson and Clifford Richardson.

In December, 1885, Francis V. Greene, M. Am. Soc. C. E., read a paper before this Society, entitled "An Account of Some Observations of Street Traffic,"† in which he presented certain data, in regard to ten of the large cities of the United States, collected in that year by the employees of the Barber Asphalt Paving Company. He also called attention to the desirability of having traffic measured systematically at frequent intervals, on a uniform system, in the leading cities of the world, for comparative purposes. Since that date no further observations have been made, as far as the writer is aware, in any of the cities considered by Captain Greene, either in the United States or abroad, with the exception of a count, made in 1896, of the number of vehicles traversing Fifth Avenue, New York City, at various points, before the removal of the granite block pavement from that street.

In November and December, 1904, additional data were collected, at the writer's suggestion, by employees of the Barber Asphalt Paving Company, on ten streets in New York City, for the purpose of determining the traffic carried by a number of representative streets at that time. The results of this count, as worked out by

* Presented at the meeting of September 19th, 1906.
† *Transactions*, Am. Soc. C. E., Vol. XV, p. 123.

employees of the company, were presented not long after in the engineering journals in comparison with the data of 1885, and with data for Paris and London cited by Captain Greene. The weights attributed to the vehicles, by those working up these data, were made to include the horse or horses, but this was not done in calculating the tonnage in Captain Greene's paper, and has not been the practice abroad. It seems, therefore, that the comparisons made at that time were not correct. All the data having since then come into the writer's possession, it has seemed worth while to present them here, in considerable detail, for future reference, and to compare the results derived from them with those obtained by Captain Greene, when calculated on the basis used by him in 1885, although they have also been worked out on a basis which, it is believed, represents more accurately the traffic carried by the streets in 1904. The data were collected with somewhat greater detail in 1904 than in 1885, as is shown in Table 1.

The observations were made by two persons, an observer and a recorder with a talley sheet. They were taken on two days (with the exception of those on Fifth Avenue, which occupied three days), from windows overlooking the various streets. They covered a period of eleven hours, from 7 A. M. to 6 P. M., being that time of day in which the principal traffic is concentrated, or one hour less than that covered in 1885.

For the purpose of determining the tonnage carried by the various streets, the writer has assigned the following weights to the different classes of vehicles, from data furnished by several of the most prominent carriage and truck builders.

Vehicle.	Weight.
Cab	1 050 lb.
Carriage	1 575 "
Automobile, electric	4 000 "
Automobile, gasoline	2 325 "
Electric truck	5 000 "
One-horse delivery wagon	1 000 "
Two-horse delivery wagon	1 500 "
Omnibus (Fifth Ave.)	3 200 "
Omnibus (streets other than Fifth Ave.)	1 550 "
Trucks, 1 to 8 tons	2 000 to 16 000 "

On this basis, the total tonnage per foot of width of street has been calculated, excluding the width of car tracks, where these exist, because the main traffic, especially on asphalt, is confined to that portion of the street lying between the tracks and the curb. This information is given in Table 2.

From the observations of 1885, Captain Greene divided the traffic into only three classes: light weight (less than 1 ton); medium weight (between 1 and 3 tons); and heavy weight (more than 3 tons). For the purpose of comparison with the data of 1885, the observations of 1904 have also been calculated on this basis, using, however, more detailed data in regard to vehicles which weigh more than 3 tons, as those of 6 and 7 tons are not uncommon now in the streets of New York. The results obtained in this way are presented in Table 3.

Having the preceding data, a comparison of the traffic of 1885 with that of 1904 can be made satisfactorily only on Fifth Avenue, the other points of observation in the two years not being the same. This comparison is given in Table 4.

It appears that on Fifth Avenue, 5 460 vehicles passed the Worth Monument in 1885, and 6 300 passed over the granite pavement in 1896; whereas, in 1904, 12 068 were observed between Thirty-third and Thirty-fourth Streets, an increase of about 120 per cent. In 1885 Captain Greene estimated the average weight per vehicle as 0.68 ton. In 1904, according to the weights which have been assumed as correct, the average weight per vehicle was 0.82 ton, or 0.83 ton on Captain Greene's basis. In 1885, 91% of the vehicles weighed less than 1 ton, while, in 1904, 79%, a much smaller number, were of this character. The traffic on this street, therefore, while still composed largely of light vehicles, has increased to an enormous extent in eight years, and the average weight per vehicle to a very considerable extent. The tonnage per foot of width, on Captain Greene's basis of weight, has increased from 94 to 251, or to 247 on the writer's basis.

Reference to Table 2 shows that, with the exception of First Avenue at Twenty-sixth Street, Fifth Avenue carries the heaviest traffic in the City of New York, although this traffic is composed of the largest percentage of light-weight vehicles. This fact, however, must be given a great deal of consideration, since a small

tonnage of heavy-weight vehicles will undoubtedly cause a greater deterioration of the pavement than a larger traffic of lighter ones.

In 1885 the traffic on Broadway, near Pine Street, amounted to 273 tons per ft. of width. No enumeration was made at this point in 1904, but at the two points where the count was made, between Eighteenth and Nineteenth Streets, on asphalt, and between Franklin and Leonard Streets, on granite block, the tonnage only reached 107.2 in the one case, and 65.5 in the other. These figures are on the Greene basis of calculation, which includes the car tracks in the width of the street, and which must be used for a comparison of 1885 and 1904 data. On the writer's basis of calculation, the tonnage at these two points amounts to 169.8 and 96.9.

These figures are exceeded by those for First, Fifth and Eighth Avenues, and for Fourth Street. It is quite surprising, therefore, to find that the Broadway traffic is not a heavy one, at the present time, as compared with that of several other streets in New York, while it has, very possibly, decreased rather than increased since 1885.

It is an interesting fact that the heaviest traffic which has been counted in New York is on First Avenue, at Twenty-sixth Street, and that it is largely made up of vehicles of a heavy class, averaging 2.18 tons in weight, only 26% being of less than 1 ton. It is also worthy of note that the pavement on this street is not of a form of construction calculated to withstand such a traffic, as it is not supported by a hydraulic concrete base.

For the purpose of comparing the traffic on the streets of New York with that upon streets in London and Liverpool, it seems worth while to present here certain data in regard to the latter, collected by Captain Greene, from the papers of Messrs. Hayward, Deacon, Howarth and Stayton. This information is given in Table 6.

It appears that the tonnage per foot run on some streets in London was far greater, even in 1873, than anything found in New York, reaching 412 on King William Street, a street of the same width as Fifth Avenue, where the vehicles average 1.06 tons in weight and 16 484 in number. On the other hand, on no street in London was the average weight of the vehicles as great as that on First Avenue. It must be remembered, however, that in London there is an enormous number of omnibuses which are not found

on New York streets, for instance, of the 10 776 vehicles passing along Piccadilly, 32% are of this description, and average 2 tons in weight, while only 56% are cabs, averaging ½ ton each. Notwithstanding this, the traffic on Fifth Avenue approaches in tonnage per foot run that on Piccadilly in 1873. Unfortunately, there are no data available for the latter street at the present time. The traffic, no doubt, has increased largely.

It has been the custom on the Continent, and, in some cases, in England, to classify the traffic according to the number of horses attached to the vehicles, or, as it is denoted in Paris, by the number of collars. For the purpose of comparing the traffic on New York streets with that on the streets of Paris, as recorded in a series of observations made in 1881 and 1882, the New York data have been calculated on the basis of the number of collars passing the different points, and this is shown in Table 5.

For comparison with these figures, Table 7 shows the traffic on prominent streets in Paris.

The preceding data show that the traffic in Paris in 1881 and 1882 was very much greater than that on the streets carrying the heaviest traffic in New York City at the present time. In fact, no traffic has been counted on any street in any city in the world which can equal that for the Rue de Rivoli in Paris in 1881, which at that time was nearly three times as great as that on Fifth Avenue to-day, the width of the two streets being practically the same. As in the case of some of the London streets, the great tonnage carried by this street is to be attributed to the large number of extraordinarily heavy omnibuses which traverse it, these vehicles being much heavier than those used in New York or London, and each counting for three collars. If to each collar a weight of 0.5 is assigned, the tonnage on this street would amount to 534 per ft. of width, which is nearly double that on Fifth Avenue.

As has been said, according to the method adopted on the Continent, in England, and by Captain Greene in 1885, no consideration was given to the weight of the horses in calculating traffic to which the pavements were subjected from the number of vehicles enumerated. Mr. W. G. Root, District Manager of the Barber Asphalt Paving Company, of New York City, in working out the data which were presented in the engineering journals in 1905, included the

weight of the animals as well as that of the vehicles, in arriving at a conclusion as to the traffic carried by the various streets. On this basis he obtained the results given in Table 8.

To the writer, the latter method of statement, if the estimate of the weight of the vehicle and animal is correct, seems much more desirable. From the point of view of one who has been employed for many years in the construction of pavements and in observations as to the causes of their deterioration, it has become evident that the latter is due much more to the impact of the horses' hoofs, especially in the case of asphalt pavements, than to vehicles rolling quietly over the surface; this, of course, with the understanding that the base supporting the pavement is sufficiently rigid to do so satisfactorily and prevent vibration in the wearing surface.

For these reasons it seems desirable to consider the data in Table 8 as representing the true traffic on the various streets in regard to which the data were calculated, and to suggest that, in any observations made in the future, due consideration should be given to the weight of the animal as well as to that of the vehicle.

TABLE 1.—TRAFFIC ON NEW YORK STREETS, 1904, 7 A. M. TO 6 P. M. AVERAGE NUMBER OF VEHICLES.

Street.		Cabs.	Auto-Vehicles.			One-horse delivery wagons.	One-ton trucks.	Carriages.	Omnibuses.	Two-horse delivery wagons	Trucks.							Total Number of vehicles.
			Elec. tric.	Gaso- line.	Electric trucks.						2-ton.	3-ton.	4-ton.	5-ton.	6-ton.	7-ton.	8-ton.	
5th Ave.,	33d to 34th Sts	6 162	838	272	74	1 399	351	1 895	370	132	323	129	68	35	8	5	7	12 068
1st Ave.,	26th to 27th Sts	185	14	7	25	1 141	1 381	10	1	122	1 332	582	351	277	180	91	241	6 037
8th Ave.,	34th to 37th Sts	417	37	37	44	2 349	928	114	5	292	720	375	209	141	40	12	49	5 769
Broadway,	18th to 19th Sts	1 005	134	58	40	1 132	402	252	3	54	332	178	93	52	20	6	10	3 771
4th St.,	Wooster St. to West Broadway	229	38	21	13	829	884	71	2	56	589	283	122	114	62	8	76	3 397
Broadway,	Franklin to Leonard Sts	392	55	44	32	1 743	243	80	2	362	207	90	16	5	1	3	2	3 277
10th Ave.,	22d to 23d Sts	62	3	2	4	1 007	458	11	0	341	548	205	118	81	17	16	19	2 892
3d St.,	Mercer to Greene Sts	24	2	2	3	513	444	6	0	46	250	132	53	51	20	4	24	1 574
2d Ave.,	34th to 35th Sts	32	3	2	6	559	206	15	2	77	169	69	30	40	6	1	8	1 225
34th St.,	Broadway to 7th Ave	97	7	15	45	395	148	52	0	49	116	45	15	19	2	0	13	1 018

TABLE 2.—TRAFFIC ON NEW YORK STREETS, 1904, 7 A. M. TO 6 P. M.

Street.		Width, in feet.	Vehicles.								Tonnage.			Collars.	
			Less than 1 ton.		From 1 to 3 tons.		From 4 to 6 tons.		From 7 to 8 tons.		Total.	Total.	Per vehicle.	Per foot of width.	Total.	Per foot of width.
			No.	%	No.	%	No.	%	No.	%						
5th Ave.,	33d to 34th Sts	40	9 588	79.45	2 357	19.54	111	0.92	12	0.09	12 068	9 875.0	0.82	246.9	16 007	400.2
1st Ave.,	26th to 27th Sts	44	1 555	25.76	3 342	55.86	808	13.38	332	5.50	6 037	13 167.8	2.18	299.2	10 150	230.7
8th Ave.,	34th to 37th Sts	44	3 172	54.98	2 146	37.20	390	6.76	61	1.06	5 769	7 683.8	1.33	174.6	8 049	18 .9
Broadway,	18th to 19th Sts	25	2 443	64.78	1 147	30.42	165	4.38	16	0.42	3 771	4 244.3	1.47	169.8	5 033	201.3
4th St.,	Wooster St. to West Broadway	32	1 185	34.88	1 830	53.88	298	8.77	84	2.47	3 397	5 772.0	1.70	180.4	5 091	159.1
Broadway,	Franklin to Leonard Sts	28	2 577	78.64	673	29.53	22	0.67	5	0.16	3 277	2 714.4	0.83	96.9	4 143	147.9
10th Ave.,	22d to 23d Sts	34	1 421	49.14	1 220	42.18	216	7.47	35	1.21	2 892	4 237.8	1.47	124.6	4 388	129.0
3d St.,	Mercer to Greene Sts	28	589	37.42	833	52.93	124	7.88	28	1.77	1 574	2 469.0	1.57	88.0	2 263	80.8
2d Ave.,	34th to 35th Sts	46	683	55.76	457	37.31	76	6.20	9	0.74	1 225	1 568.5	1.28	34.1	1 706	37.1
34th St.,	Broadway to 7th Ave	24	593	58.25	376	36.93	36	3.54	13	1.28	1 018	1 251.1	1.23	52.1	1 415	58.9

NOTE.—The width does not include the car tracks, as these carry only a small proportion of the traffic.

TABLE 3.—Traffic on New York Streets, 1904, 7 A. M. to 6 P. M.

Street.		Width, in feet.	Vehicles.								Total.	Tonnage.			Collars.	
			Less than 1 ton.		From 1 to 3 tons.		From 4 to 6 tons.		From 7 to 8 tons.			Total.	Per vehicle.	Per foot of width.	Total.	Per foot of width.
			No.	%	No.	%	No.	%	No.	%						
5th Ave.,	33d to 34th Sts.	40	9 588	79.45	2 357	19.54	111	0.92	12	0.09	12 068	10 036.0	0.83	250.9	16 007	400.2
1st Ave.,	26th to 27th Sts.	44	1 555	25.76	3 342	55.86	808	13.38	332	5.50	6 037	13 017.5	2.16	295.9	10 150	230.7
8th Ave.,	34th to 37th Sts.	44	3 172	54.98	2 146	37.20	390	6.76	61	1.06	5 769	7 865.0	1.36	178.8	8 049	182.9
Broadway,	18th to 19th Sts.	25	2 443	64.78	1 147	30.42	165	4.38	16	0.42	3 771	4 287.5	1.14	171.5	5 033	201.3
4th St.,	Wooster St. to West Broadway	32	1 185	34.88	1 830	53.88	293	8.77	84	2.47	3 397	6 032.5	1.78	188.5	5 091	159.1
Broadway,	Franklin to Leonard Sts.	28	2 577	78.64	673	20.53	22	0.67	5	0.16	3 277	2 752.5	0.84	98.3	4 143	147.9
10th Ave.,	22d to 23d Sts.	34	1 421	49.14	1 220	42.18	216	7.47	35	1.21	2 892	4 259.5	1.47	125.3	4 388	129.0
3d St.,	Mercer to Greene Sts.	28	589	37.42	833	52.93	124	7.88	28	1.77	1 574	2 652.5	1.69	94.7	2 263	80.8
2d Ave.,	34th to 35th Sts.	46	683	55.76	457	37.31	76	6.20	9	0.74	1 225	1 622.5	1.32	35.3	1 706	37.1
34th St.,	Broadway to 7th Ave.	24	593	58.25	376	36.93	36	3.54	13	1.28	1 018	1 283.5	1.26	53.5	1 415	58.9

Note.—The width does not include the car tracks, as these carry only a small proportion of the traffic.

TABLE 4.—Traffic on New York Streets: Comparison of 1885 and 1904.

Year.	Street.		Width, in feet.	Pavement.	Vehicles.						Total.	Tonnage.		
					Less than 1 ton.		From 1 to 3 tons.		Over 3 tons.			Total.	Per vehicle.	Per foot of width.
					No.	%	No.	%	No.	%				
1885	Broadway,	near Pine St.	40	Granite	4 379	54	2 502	34	930	12	7 811	10 905	1.39	273.0
	5th Ave.,	opposite Worth Monument	40	Asphalt.	4 957	91	373	7	130	2	5 460	3 744	0.68	94.0
1904	Broadway,	18th to 19th Sts.	40*	Asphalt.	2 443	64.78	1 147	30.42	181	4.80	3 771	4 287.5	1.14	107.2
	Broadway,	Franklin to Leonard Sts.	42*	Granite	2 577	78.64	673	20.53	27	0.83	3 277	2 752.0	0.84	65.5
	5th Ave.,	33d to 34th Sts.	40	Asphalt.	9 588	79.45	2 357	19.54	123	1.04	12 068	10 036.0	0.83	250.9

* In 1904 the width of the streets has been taken exclusive of that of the car tracks, as these carry but a small portion of the traffic. For the purpose of comparison with the data of 1885 the width of Broadway has been considered as including the car tracks in this table.

TABLE 5.—Traffic on New York Streets, 1904, 7 A. M. to 6 P. M., by Collars.

Street.	Width, in feet.	Vehicles.							Collars.	
		One-horse.		Two-horse.		Three-horse and over.		Total.	Number.	Per foot of width.
		No.	%	No.	%	No.	%			
5th Ave., 33d to 34th Sts.	40	8 184	67.8	3 829	31.7	55	0.5	12 068	16 007	400.2
1st Ave., 26th to 27th Sts	44	2 714	44.9	2 534	42.0	789	13.1	6 037	10 150	230.7
8th Ave., 34th to 37th Sts.	44	3 731	64.7	1 796	31.1	242	4.2	5 769	8 049	182.9
Broadway, 18th to 19th Sts.	25	2 597	68.9	1 086	28.8	88	2.3	3 771	5 033	201.3
4th St., Wooster St. to West Broadway.	32	1 963	57.8	1 174	34.6	260	7.6	3 397	5 091	159.1
Broadway, Franklin to Leonard Sts.	28	2 422	73.9	844	25.8	11	0.3	3 277	4 143	147.9
10th Ave., 22d to 23d Sts	34	1 529	52.9	1 230	42.5	133	4.6	2 892	4 388	129.0
3d St., Mercer to Greene Sts.	28	983	62.4	492	31.3	99	6.3	1 574	2 263	80.8
2d Ave., 34th to 35th Sts.	46	799	65.2	371	30.3	55	4.5	1 225	1 706	37.1
34th St., Broadway to 7th Ave.	24	655	64.4	329	32.3	34	3.3	1 018	1 415	58.9

1 Collar = Cabs, Gasoline Autos, 1-Horse Delivery Wagons and 1-Ton Trucks.
2 Collars = Carriages, Omnibuses, 2-Horse Delivery Wagons, Electric Trucks, Electric Autos, and 2, 3 and 4-Ton Trucks.
3 Collars = 5, 6, 7 and 8-Ton Trucks.

TABLE 6.—Traffic Records on Certain Streets in London and Liverpool, 1873 to 1884.

City.	Street.	Pavement.	Width, in feet.	Total number of vehicles.	Tonnage.		
					Total.	Per vehicle.	Per foot of width.
London....	Gracechurch St....	Asphalt....	32	12 148	13 507	1.11	422
"	King William St...	Wood	40	15 513	16 484	1.06	412
"	The Poultry.....	Asphalt....	22	8 167	8 330	1.02	378
"	Strand and Fleet St.	Wood	37	16 208	13 596	0.84	367
"	Parliament St.....	Macadam..	45	14 306	14 380	1.01	322
"	Oxford St.....	Wood.....	57	16 886	17 076	1.01	300
"	Cheapside.....	Asphalt....	32	9 419	9 260	0.98	290
"	Leadenhall St.....	"	30	6 128	7 588	1.08	253
"	Piccadilly.....	Macadam..	37	10 776	9 358	0.87	253
"	Euston Road.....	Granite	44	12 132	10 658	0.88	242
"	Brompton Road...	Wood.....	.....	.....	.....	.....	216
"	King William St...	Granite....	32	6 371	6 506	1.02	203
"	Edgeware Road...	"	43	8 212	8 376	1.02	195
"	Regent St.....	Macadam..	52	10 796	9 668	0.90	186
"	King's Road.....	Wood.....	.....	.....	.....	.....	156
"	Victoria St.....	Macadam..	40	6 040	5 780	0.96	145
"	Sloane St.....	Wood.....	.....	.....	.....	.....	93
Liverpool..	.....	Granite....	.....	.....	.....	.....	382
" ..	.....	"	.....	.....	.....	.....	232
" ..	Great Howard St..	Wood.....	.....	.....	.....	.....	231
" ..	Bold St.....	"	.....	.....	.....	.....	100

TABLE 7.—Traffic on Streets in Paris, France, 1881-1882, by Collars.

Street.	Width, in feet.	Number of Collars.	Collars, per foot of width.	Total number of vehicles.
Avenue de l'Opera.....	52.48	36 185	689.5	29 460
Rue de Rivoli.....	39.36	42 035	1 067.9	33 232
Rue Saint-Honore.....	28.208	19 672	697.4	16 589
Boulevard Haussmann.....	45.92	14 096	306.9	12 638
Avenue des Champs Elysées	89.872	14 082	156.7	12 023

TABLE 8.—Traffic on New York Streets, 1904, 7 A. M. to 6 P. M., Weight of Animals included with Vehicles.

Street.		Average tonnage per vehicle.	Average tonnage per linear foot of width for 11 hours.
5th Ave.,	33d to 34th Sts.....	1.64	481.85
1st Ave.,	26th to 27th Sts.....	3.18	435.58
8th Ave.,	35th to 36th Sts.....	2.28	296.02
Broadway,	18th to 19th Sts.....	1.97	299.63
4th St.,	Wooster St. to West Broadway.....	2.73	289.18
Broadway,	Franklin to Leonard Sts.....	1.74	202.42
10th Ave.,	22d to 23d Sts.....	2.69	227.45
3d St.,	Mercer to Greene Sts.....	2.51	140.50
2d Ave.,	34th to 35th Sts.....	2.24	117.86
34th St,	Broadway to 7th Ave.....	2.03	89.22

DISCUSSION.

S. WHINERY, M. AM. SOC. C. E. (by letter).—Records of street travel, such as those given by the author, possess a general interest to the student of urban life and development. To the municipal engineer, their chief, and practically their only, value consists in the evidence they afford of the service and endurance of street pavements. Mr. Whinery.

While the relative durability of the several kinds of pavement used on the streets of American cities is by no means the controlling factor in comparing the merits of one pavement with another, it is yet one of very great importance. Considering its importance, it is surprising how few reliable and satisfactory data there are upon the subject. More capital is invested in street pavements than in any other department of municipal work, and the charges for pavement maintenance constitute, usually, the largest annual expenditure for any one public utility in municipal accounts. It would naturally be expected, therefore, that the relative economy of different street pavements would be studied carefully by municipal engineers, and that such experimental investigations would be conducted as would furnish the necessary data for an intelligent discussion of the subject.

It is now generally accepted among engineers—though not yet appreciated by the public—that, other things being equal, the economic life of a pavement is measured by the quantity of travel it is capable of enduring before it is worn out, and not by its age in years. This being so, the important thing is to ascertain, as accurately as may be practicable, the service record of the various kinds of pavement in use, measured by some common standard which will permit of intelligent comparison of results. Here, as in other departments of scientific investigation to determine relative values, isolated observations that are not based upon, or not reducible to, uniform standards are comparatively worthless. However interesting they may be, and whatever they may suggest, they lack the one element that would make them practically useful. Unfortunately, this may be said of nearly all the few observations that have been made on street travel. Aside from their inadequateness, they are not comparable upon a common basis.

Until it is generally recognized that there is the same necessity for uniform methods and standards in this work as in other scientific and engineering investigation—as for instance in the testing of structural materials and the performance of boilers and engines—street censuses of travel will continue to possess very little engineering value.

The writer has long felt that the value to municipal engineers of adequate and properly conducted censuses of street travel would be

Mr. Whinery. so great as to make their cost a trifling consideration. In no other way can the travel-endurance value of the several kinds of pavement, under the varying conditions of actual service, be even approximately determined.

It is a field in which the municipal engineer finds at present no reliable data to guide him in one of the most important departments of his work. If it were conclusively established that of two varieties of pavement, approximately equal in other respects, one is capable of 25% greater endurance under travel, or, in other words, that its economic life was one-quarter greater than the other, the knowledge might well be worth hundreds of thousands of dollars to our cities.

The writer does not desire to discuss the subject in detail here, but wishes to call attention, first, to the very great value of properly conducted street censuses of travel, and second, to the importance of the adoption of standard units and methods of conducting such work. It would be well worth while for some technical society dealing with municipal engineering to appoint a committee to devise and formulate a standard code of rules and regulations for the guidance of those who may have opportunity or occasion to engage in such investigations.

Mr. Lewis. N. P. Lewis, M. Am. Soc. C. E.—It is fortunate that Mr. Richardson has presented this paper to the Society, for, while interest in the subject may be largely academic, such investigations, as pointed out by Mr. Whinery, will have a very practical value in their application to the study of the efficiency and wearing qualities of different kinds of pavement.

It seems remarkable that so little work of this kind has been attempted; but the engineers in responsible charge of the highways of large cities are exceedingly busy men, and have very little time for investigations of professional interest and value such as they would like to make. It is also strange that so little has been done in the United States to confirm or elaborate the observations made by General Greene in 1885.

It is amusing to see how writers on the subject of highways and their improvement have been copying and incorporating in their books and papers, year after year, the old tables showing the results of the observations of Hayward and others of the traffic on London streets, and on those of Paris—observations which were made a generation or more ago. These old figures of the thirty odd thousand vehicles on the Rue de Rivoli have a very familiar look, and may be found in almost every book on the subject. Whether they have ever been verified the speaker does not know, but Mr. Richardson has stated that he made some observations, or had some made, recently in Paris, and that the number of vehicles did not come up to the old figures given for the Rue de Rivoli.

Mr. Lewis.

It will be noticed that the observations described by Mr. Richardson extended over two days on the New York streets, with the exception of those on Fifth Avenue, which were continued for three days. He referred to the practice in Great Britain of taking observations for one hour on one day, another hour on the next day, at still another hour on the third, and so on. In the speaker's judgment, the most satisfactory and accurate results would be obtained by making observations, not on one day or two days—certainly not at different hours on succeeding days—but by continued observations extending over not less than one week.

Observations of that kind were taken on a number of streets in Brooklyn, N. Y., in January, 1896, and demonstrated very clearly the erroneous deductions which might be made from observations limited to one day or to a couple of days—certainly from those covering but an hour or two.

While the results of these observations (presented graphically in Fig. 1) have no special interest on account of the volume of traffic, they illustrate forcibly the sudden changes which may occur in the volume of such traffic, especially on asphalt pavements. It will be seen that, under normal conditions, there are noonday and afternoon maxima. On Monday, January 6th, which was an unusually cold day, with an average temperature of 4°, the volume of traffic was somewhat irregular, with a decrease between 10 and 11 A. M., in addition to the customary lull which always occurs at about 1 P. M. On Tuesday, as the temperature rose the travel increased, but at 1.45 P. M. snow began to fall, and the line indicating the volume of traffic immediately fell in the most striking manner. Again, on Saturday, the snow had nearly all disappeared from the center of the roadway by noon, and it will be seen that, for the first time since the snow appeared, there was an afternoon maximum.

It may be said that the variations indicated by this diagram are those of an extreme case, but they illustrate forcibly how the volume of traffic on asphalt or other smooth pavements is affected by local conditions, and lead one to question the accuracy of results obtained from observations lasting only one or two days.

The classification of vehicles made by the author is quite minute, including vehicles of less than 1 ton, those from 1 to 3 tons, from 4 to 6 tons, and from 7 to 8 tons. In the observations made in Brooklyn, a simpler classification was used, but all horses were included, whether they were driven, ridden or led, and a careful record was kept of the falls, which was also a conspicuous feature of Mr. Hayward's London observations. These falls were classified as falls on the knees, falls on the haunches, and complete falls.

The author's data appear to have been collected by two men, an observer and a recorder. The observer, therefore, was obliged to

Mr. Lewis. assign to one of at least four classes more than 12 000 vehicles passing before him on Fifth Avenue in 11 hours; but the rate was by no means uniform, and it is more than probable that the rate of their passage at certain hours reached that of 2 000 or even 3 000 an hour. The recorder was also obliged to assign each of these vehicles to its proper column. This was an exceedingly difficult thing to do with any degree of accuracy, and, doubtless, there was a great deal of guessing and a large number of errors.

It is difficult to explain the extraordinary falling off in the traffic on Broadway, New York City. In 1885 this traffic was about 7 800 vehicles per day at Pine Street, while in 1904 it was found

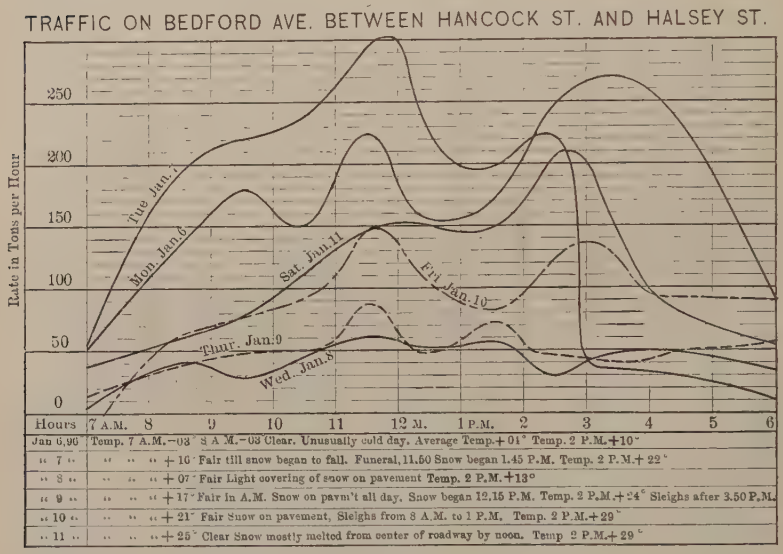

TRAFFIC ON BEDFORD AVE. BETWEEN HANCOCK ST. AND HALSEY ST.
Rate in Tons per Hour
250
200
150
100
50
0
Tue. Jan.7
Mon. Jan.6
Sat. Jan.11
Fri. Jan.10
Thur. Jan.9
Wed. Jan.8
Hours 7 A.M. 8 9 10 11 12 M. 1 P.M. 2 3 4 5 6
Jan 6,96 Temp. 7 A.M.−03° 8 A.M.−03 Clear. Unusually cold day. Average Temp.+01° Temp. 2 P.M.+10°
" 7 " " " " +16° Fair till snow began to fall. Funeral, 11.50 Snow began 1.45 P.M. Temp. 2 P.M.+22°
" 8 " " " " +07° Fair Light covering of snow on pavement Temp. 2 P.M.+13°
" 9 " " " " +17° Fair in A.M. Snow on pavm't all day. Snow began 12.15 P.M. Temp. 2 P.M.+24° Sleighs after 3.50 P.M.
" 10 " " " " +21° Fair Snow on pavement, Sleighs from 8 A.M. to 1 P.M. Temp. 2 P.M.+29°
" 11 " " " " +25° Clear Snow mostly melted from center of roadway by noon. Temp 2 P.M.+29°

FIG. 1.

to be less than 3 300 per day between Franklin and Leonard Streets, and 3 771 per day betwen 18th and 19th Streets. This indicates a decrease of more than 50% in the number of vehicles using Broadway.

Possibly this can be explained by the fact that the side streets of lower Manhattan now have very much better pavements than they had twenty years ago, while the decrease of traffic in Broadway between Franklin and Leonard Streets might be largely accounted for by the widening and improvement of Elm Street; but, that there has been a decided decrease in the number of vehicles and the tonnage on Broadway appears to have been pretty well demonstrated.

Mr. Lewis.

In order that statistics of street travel may have any real value, a standard method of observation and compilation is most necessary. The committee on Standard Tests of Road Materials, of the American Society for Testing Materials, has discussed the desirability of securing data on street traffic, but, although the speaker was a member of the sub-committee which intended to investigate this subject, he has been unable to attend the conventions, and is not aware of any definite action which may have been taken.

The speaker would again emphasize the importance of such observations as the author has made, and his appreciation of their value in engineering investigations of pavements and roads, and the materials and methods used in their construction and maintenance.

Mr. Tillson.

G. W. Tillson, M. Am. Soc. C. E.—It is exceedingly interesting to find printed information about street pavements. Engineering literature is so wofully deficient in data regarding the forces that tend to destroy pavements that anything of the kind is doubly welcome to the engineer who is making specifications for or constructing them.

It seems to be a fact that, in making specifications for or laying street pavements, nearly all engineers are now using an old rule-of-thumb, the same as used forty or fifty years ago. An understanding of this subject, of course, is more necessary in designing asphalt than other pavements, and the author is about the only man who has been studying the matter systematically, and applying his knowledge in such a way that he designs a pavement to meet the traffic conditions on the street.

There are a great many difficulties in obtaining specific information of this kind. The way in which work is carried on—most of it by contract—does not give cities an exact knowledge of the cost of different pavements. A short time ago an attempt was made to decrease the expense of the pavement on some streets where it was known that the traffic would be light. To do this the wearing surface of the pavement—an expensive one—was decreased 12½ per cent. The result was that the price bid was just as high as when the full amount of wearing surface was required. It should be stated that this wearing surface was practically furnished by one party, and there was very little competition. The specifications were then changed, the full depth of wearing surface being stipulated, as it was thought that, if the full amount was to be paid for, the full depth should be obtained.

In studying these statistics, it is difficult to get the facts, in order to know the effect on the pavements, and some of the data in the paper illustrate this very well. The number of vehicles reported on Fifth Avenue was 12 000, and their tonnage a little more than 9 000,

Mr. Tillson. which gives a foot-tonnage of 246. On First Avenue, where the total number of vehicles was 6 000, or half as many as on Fifth Avenue, the tonnage was 13 000, or a little more than 299 tons per foot. From this it might be said that the two streets would require nearly the same kind of pavement; but, the character of the traffic should be examined. Of the 12 000 vehicles reported on Fifth Avenue, 50% were cabs with rubber tires, and each was drawn by one horse; 12% were automobiles with rubber tires, and were driven by their own power. On First Avenue, 3% of the entire number of vehicles were cabs, and 1% was automobiles. It can be seen, therefore, that if the effect of a 2-ton automobile on a pavement is compared with the effect of a 2-ton truck, and the same consideration is given to each, that the variable may be large.

It has always seemed to the speaker that a 2-ton vehicle drawn over a rigid pavement—such as granite or brick—would do much more damage if drawn at, say, 10 miles per hour than would a 4-ton truck if drawn at the rate of 3 miles per hour.

About eight years ago, Kent Avenue, in Brooklyn, was paved with granite on a concrete base, the blocks being the hardest that come to the New York market. This avenue is on the river front where one side of the street is lined with sugar packing-houses, and it was estimated at that time that 1 200 sugar trucks, each weighing, with its load, 8 or 10 tons, were driven over this pavement every day. All these trucks had to return, and, taking their weight at 2 or 3 tons, which is not excessive, it gives a tonnage of 14 000 or 15 000; and, as the roadway is only 34 ft. wide, including street-car tracks, the tonnage was 450 tons per foot. Now, compare 450 tons on Kent Avenue, considering the character of its traffic, with 450 tons of the kind of traffic on Fifth Avenue, and it will be seen that the variable may be large.

Mr. Whinery's suggestion, to adopt some systematic method of taking these observations, is a very good one. In the United States, officials have been very backward in work of this kind, and it is a sad commentary on the foresight or enterprise of municipal corporations when the only information to be obtained regarding street traffic is that furnished by a commercial corporation. The only case known to the speaker where a municipal corporation attempted to investigate its traffic needs is that mentioned by Mr. Lewis.

There was one interesting feature of those needs which, perhaps, he forgot to mention, but which illustrates what is often found if the traffic on certain days and at certain hours is taken as the standard. On a certain day the curve ran very high, and upon inquiry it was learned that a very large funeral passed through the street at about that time.

CLIFFORD RICHARDSON, ASSOC. AM. SOC. C. E. (by letter).—Since the appearance of this paper, the writer has been privileged to spend five weeks in London, where he made some personal observations of the traffic, obtained many interesting data,* and became acquainted with a publication on the subject by Mr. Edgar Harper. In an article† entitled "Statistics of London Traffic," that gentleman has collected and published some extremely interesting data on the subject of vehicular traffic in the Strand, Holborn, and Piccadilly. Mr. Richardson.

Additional data have also been collected in reference to traffic on the streets of New York, Boston and Chicago.

It appears that in the Strand, from 8 A. M. to 8 P. M., on July 4th, 1902, 11 982 vehicles passed, while more than a year later, on September 18th, 1903, during the same hours, there were 12 385. Of these vehicles, rather more than 4 000 were omnibuses.

In Holborn, for about the same periods, the total numbers of vehicles were 11 095 and 11 727, the number of omnibuses falling slightly below 3 000.

In Piccadilly, at Devonshire House, on a day in January, 1903, between 8 A. M. and 8 P. M., 16 064 vehicles passed, while on September 2d, 1903, only 13 875 passed; about 4 500 of these were omnibuses. Mr. Harper cites the fact that in July, 1865, only 8 220 vehicles passed the same point, during the same hours, thus showing a great increase of traffic.

Some of the interesting data from this report are contained in Table 9.

On March 22d, 1906, an examination, extending over a period of 3 hours, was made of the traffic on the Thames Embankment, near Charing Cross, which showed that 2 158 vehicles passed, or at the rate of 8 632 vehicles for a period of 12 hours. Of this traffic, more than one-half were cabs and carriages, the remainder being vans, motors and carts.

On July 18th, 1906, the traffic on the Embankment, at Temple Avenue, as counted by the writer, amounted to 652 light vehicles and 328 heavy vans, etc., between the hours of 10 and 11 A. M., or at the rate of 11 760 vehicles in 12 hours. Between 4 and 5 P. M. 888 vehicles passed, or at the rate of 10 665 in 12 hours. The actual number passing during a 12-hr. period is undoubtedly much smaller, as the hours at which the count was made are those in which the traffic on the Embankment is greatest. It is probable, therefore, that the count of March 22d, 1906, represents more nearly the actual traffic on this roadway, which is constructed of macadam and is very difficult to maintain because of the heavy traffic.

The traffic on Piccadilly was counted by the writer on two

* Contained in Vol. 7 and 8, "Report of the Royal Commission on London Traffic."

† *Journal* of the Royal Statistical Society, Vol. 67, June 13th, 1904.

TABLE 9.—Comparison of Omnibus and Other Vehicular Traffic, in London.*

Hour.	Number of Vehicles (Exclusive of Barrows and Cycles), Passing Both Ways Along:											
	The Strand (At Wellington Street).				Holborn (At Southampton Row).				Piccadilly (At Devonshire House).			
	Omnibuses.	Other vehicles.	Total.	Percentage of omnibuses to total vehicles.	Omnibuses.	Other vehicles.	Total.	Percentage of omnibuses to total vehicles.	Omnibuses.	Other vehicles.	Total.	Percentage of omnibuses to total vehicles.
8 to 9 A. M.	202	398	600	33.6	181	361	542	33.4	334	308	642	52.0
9 to 10 "	365	538	903	40.4	270	465	735	36.7	400	496	896	44.6
10 to 11 "	358	683	1 041	34.4	255	591	846	30.1	382	674	1 056	36.2
11 to 12 noon	328	673	1 001	32.8	239	573	812	29.4	387	885	1 272	30.4
12 noon to 1 P. M.	350	637	987	35.5	250	608	858	29.1	374	1 060	1 434	26.1
1 to 2 P. M.	329	561	890	37.0	245	557	802	30.5	366	1 099	1 465	25.0
2 to 3 "	344	575	919	37.4	243	552	795	30.6	384	937	1 321	29.1
3 to 4 "	359	600	959	37.4	256	560	816	31.4	387	1 007	1 394	27.8
4 to 5 "	369	677	1 046	35.3	239	617	856	27.9	369	1 026	1 395	26.5
5 to 6 "	360	618	978	36.8	284	601	885	32.1	399	1 339	1 738	23.0
6 to 7 "	356	517	873	40.8	247	569	816	30.3	394	905	1 299	30.3
7 to 8 "	333	560	893	37.3	242	414	656	36.9	399	973	1 372	29.1
Totals	4 053	7 087	11 090	36.5	2 951	6 468	9 419	31.3	4 575	10 709	15 284	29.9

* From a paper by Mr. Edgar Harper, *Journal*, Royal Statistical Society, Vol. 67, June 13th, 1904.

Mr. Richardson.

occasions. Between 4 and 5 P. M. on July 17th, 1906, 1 344 vehicles passed Dover Street, or at the rate of 16 128 for a period of 12 hours. On July 19th, 1906, between 11.30 and 12.30 A. M., the count showed: Mr. Richardson.

Motor omnibuses	108
Motors	124
Cabs	1 412
Ordinary omnibuses	480
	2 124

or at the rate of 25 488 vehicles in 12 hours.

These counts, like those on the Embankment, being made at the busiest time of the day, the daily traffic probably does not reach the high figures given for 12 hours, but it must be remembered that, in the fashionable season, the traffic on Piccadilly is extremely heavy at all hours of the day, and much greater than in January or September when the count given by Mr. Harper was made.

Mr. Harper has made no calculation of the tonnage of the traffic passing on London streets, but it is of interest to compare the data as to the number of vehicles, collected by him and by the writer, with the data for New York streets collected in 1904. In this respect, the traffic on Fifth Avenue is about as large as that on Piccadilly, as far as the number of vehicles is concerned, and is larger than that in the Strand or Holborn. If consideration is given to the fact that the traffic of London streets is largely made up of omnibuses, there can be no comparison as to the tonnage of the two, as the London traffic is much heavier.

From the point of view of the number of collars, the traffic on the two streets is, probably, very similar, and the wear and tear due to the impact of the horses' hoofs is quite the same.

It may be of interest to remark here that the wood pavement on Piccadilly is replaced every five or six years, under this traffic, the blocks being removed when they have been reduced to a thickness of 3 in.

A very elaborate determination of the traffic of the various streets in London has been made under the auspices of the Royal Commission on London Traffic, and summaries of some of the most interesting results are presented in Tables 10 and 11.

The data in Table 10 are confirmative of the data in Mr. Harper's paper. Those presented in Table 11 show that at certain representative crossings the traffic is enormously greater, reaching nearly 30 000 vehicles in a period of 12 hours at Marble Arch and Hyde Park Corner. There is, of course, nothing in the United States to correspond to this, as far as known. The traffic in New York has not been taken at the busiest time of the year, at such in-

Mr. Richardson.

TABLE 10.—Traffic in Selected Streets.*

Street.	Goods Vehicles.				Passenger Vehicles.				Cycles and Barrows.				Total Vehicles of All Classes.		
	Day: 8 A. M. to 8 P. M.		Busiest hour.		Day: 8 A. M. to 8 P. M.		Busiest hour.		Day: 8 A. M. to 8 P. M.		Busiest hour.		Day: 8 A. M. to 8 P. M.	Busiest hour.	
	Number.	Per cent. of total.	Number.	Per cent. of total.	Number.	Per cent. of total.	Number.	Per cent. of total.	Number.	Per cent. of total.	Number.	Per cent. of total.	Number.	Number.	Per cent. of total during day.
Bayswater Road (West of Edgeware Road)	2 832	24.0	181	14.7	7 274	61.5	817	66.3	1 710	14.5	234	19.0	11 816	1 232	10.4
Oxford Street (East of Regent Street)	3 778	27.0	349	26.0	8 192	58.5	913	67.7	2 043	14.5	85	6.3	14 013	1 347	9.6
Cheapside (East of St. Paul's Church Yard.)	4 436	35.3	505	38.2	6 108	48.6	600	45.4	2 019	16.1	217	16.4	12 563	1 322	10.5
Adgate High Street (East of Minories)	6 943	64.5	719	70.3	1 796	16.7	165	16.1	2 019	18.8	139	13.6	10 758	1 023	9.5
Piccadilly (East of Hamilton Place)	3 118	22.3	249	18.3	9 616	68.6	974	71.6	1 278	9.1	136	10.1	14 012	1 359	9.7
Fleet Street (West of Ludgate Circus)	2 916	32.7	282	32.8	4 933	55.2	487	56.6	1 083	12.1	91	10.6	8 932	860	9.6
Victoria Embankment (West of Blackfriars Bridge)	2 875	31.7	278	27.6	5 266	58.1	675	67.0	922	10.2	54	5.4	9 063	1 007	11.1
Queen Victoria Street (East of Cannon Street)	3 382	27.4	379	28.4	7 780	63.2	870	65.3	1 157	9.4	83	6.3	12 319	1 332	10.8

* From "Report of Royal Commission on London Traffic." Table 6, Vol. 7, 1905, page 11.

tersections as Fifth Avenue, Broadway and Twenty-third Street, at Thirty-fourth Street, Sixth Avenue and Broadway; Thirty-fourth Street and Fifth Avenue; or Thirty-fourth Street and First Avenue. During the quiet months of July and August, counts were made at these places, the results being given in Table 12. Mr. Richardson.

TABLE 11.—Number of Vehicles Using Eight Representative Crossings.*

Points of observation.	Number of vehicles.	
	Per day of 12 hours. 8 A. M. to 8 P. M.	At busiest hour.
Marble Arch	(1) 29 320	(1) 3 069
Hyde Park Corner	(2) 29 286	(2) 2 902
Piccadilly, at St. James	(7) 20 474	(7) 2 067
Piccadilly Circus	(5) 27 050	(4) 2 670
Charing Cross	(3) 27 768	(5) 2 642
Strand, at Wellington Street	(8) 19 743	(8) 1 821
Ludgate Circus	(6) 22 956	(6) 2 252
Bank	(4) 27 523	(3) 2 699

* From "Report of Royal Commission on London Traffic." Table 39, Vol. 7, 1905, page 55.

TABLE 12.—Traffic on New York Streets—1906.

Date.	Hour.	Street.	Vehicles.		Half hour.	12 Hours.	Weather.
			Trucks.	Cabs, Autos., etc.			
8-13-06	9.41-10.24 A. M.	Fifth Avenue and Thirty-fourth Street.	125	196	321	7 704	Fair.
8-13-06	3.05-3.35 P. M.		155	291	446	10 704	Fair.
8-14-06	3.45-4.15 "		141	277	418	10 032	Fair.
8-16-06	4.15-4.35 "		132	298	430	10 320	Fair.
8-24-06	2.35-3.05 "		112	242	364	8 736	Threatening.
9- 6-06	4.35-5.05 "		189	364	553	13 272	Fair.
8-24-06	3.15-3.45 "	Sixth Avenue, Broadway and Thirty-fourth Street.	306	192	498	11 952	Threatening.
9- 6-06	2.15-2.45 "		263	197	460	11 040	Fair.
8-24-06	4.05-4.35 "	Fifth Avenue, Broadway and Twenty-third Street	240	195	435	10 440	Threatening.
9- 6-06	3.02-3.32 "		201	280	481	11 544	Fair.
9- 6-06	3.55-4.25 "	First Avenue and Thirty-fourth Street.	342	110	452	10 848	Fair.
8-13-06	10.50-11.20 A. M.	Eighth Avenue and Forty-first Street.	185	22	207	4 968	Fair.
8-13-06	3.50-4.20 P. M.		170	28	198	4 752	Fair.
8-14-06	2.53-3.23 "		156	32	188	4 512	Fair.
8-16-06	3.30-4.00 "		147	37	184	4 416	Fair.

Mr. Richardson.

In a paper* read before the Commercial Club, of Chicago, in 1904, John W. Alvord, M. Am. Soc. C. E., gives some data in regard to traffic on streets in that city. The largest amount, as recorded by him, appears to be on State Street, where it is slightly in excess of 6 000 vehicles in 12 hours, and at times does not reach 4 000, as can be seen from the following figures:

TRAFFIC ON STATE STREET, CHICAGO, ILL. (ROADWAY 60 FT.).

Date.	Hour.	Vehicles in 12 hours.
10-2-03	9.15- 9.30	6 048
3	11.45-12.00	5 952
"	10.35-11.35	3 876

In Boston a recent count of the traffic by collars on several of the most heavily traveled streets gave the results shown in Table 13.

From these data it appears that the traffic in Boston on Devonshire Street, which is the heaviest one that was counted, is very much less than that on Fifth Avenue in New York, although the street itself is much narrower.

It is proposed to continue the counts on all the streets which have been mentioned in this paper, and especially to determine the extent of the traffic at the prominent intersections in New York City.

TABLE 13.—TRAFFIC IN BOSTON, MASS., IN 1906.

Date.	Street.	Number of Hours.	Collars.
4-23-06	Devonshire Street, Milk to Water Streets	10	4 384
24	" " " " "	10	4 310
25	Tremont Street, Scollay Square to School Street	10	4 261
9-5-06	Charles Street, Fruit and Allen Streets	10	2 650
6	Court Street, Washington Street to Court Square	10	3 290
7	Columbus Avenue, Ferdinand to Berkeley Streets	9	1 440

In reply to the statements of Messrs. Whinery, Lewis and Tillson: Nobody can appreciate more than the writer the necessity for standardizing methods of counting traffic and stating the results. Unfortunately, nothing of this kind has been done, and, as a result, vehicles alone are counted in one city, vehicles and tons per foot run, either with or without the horses, in another city, and collars alone in another. As far as the writer has been able to learn, the subject has not been considered by any of the societies which might have done so, and it seems to be necessary

* "The Street Paving Problem of Chicago."

Mr. Richardson.

that something of this sort should be done, as the data which have been collected heretofore are of much less value than if they had been obtained by following some definite system.

The writer is informed by the Chairman of the Committee on Standard Tests of Road Materials, of the American Society for Testing Materials, that, as far as he is aware, that committee has never given any consideration to, or appointed any sub-committee to consider, a standard method for observing traffic and compiling data.

Mr. Lewis presents the objection that counts covering but an hour or two may lead to erroneous deductions, but determinations made in this way in London on several occasions confirm closely the results of the count of the Royal Commission extending over periods of a week or more. Of course, judgment must be used in interpreting the results and in selecting a time when an average traffic is going on.

Mr. Lewis is mistaken in supposing that the data which have been presented in the paper were collected by two men. There were four, one observer and one man with a tally sheet, for traffic in each direction. Under these circumstances, there does not seem to be reason to believe that there are any gross errors.

The reason given by Mr. Lewis as a possible cause for the falling off in the traffic on Broadway seems to the writer to be correct.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1032.

THE NARRAGANSETT BAY COAL DEPOT.*

By Augustus Smith, M. Am. Soc. C. E.

With Discussion by E. H. Foster, M. Am. Soc. C. E.

To every newspaper reader who followed Admiral Rojestvensky's progress to the Fushima Straits, the preponderating importance of coaling ship, and the seemingly continuous performance of it must have been strikingly apparent. To the men intrusted with the upbuilding of our Navy, this problem, then, ever since the retirement of the sailing man-of-war, has been of increasing importance.

It was at first deemed sufficient to depend on commercial stocks of coal in the various ports visited by our vessels, and on commercial equipments for supplying coal to the fleet. Then, as the Navy expanded and the service became more dependent on the coal pile, it seemed to be desirable for the Government to have depots containing a stock of coal which could be relied on without reference to commercial conditions, other than those which might affect the price of coal delivered at the Atlantic seaboard.

Up to this point the United States only followed the lead of Great Britain. After the establishment of these depots was started, the wisdom of providing some form of coal-handling machinery, so

* Presented at the meeting of October 3d, 1906.

as to reduce to a minimum the cost of storing and withdrawing coal for distribution, and, more important from a naval standpoint, so as to reduce the time required for these operations, impressed itself on the minds of influential men in the Navy; and, in this application of labor-saving machinery, the United States Navy to-day stands foremost.

This undertaking means more, perhaps, to those who are well informed as to the state of the art of construction of coal-handling machinery than to the more superficial observer. The commercial machinery available had been developed to handle large quantities of coal daily, or at least at frequent intervals, and, in the case of many types, it required for its successful manipulation a crew of specially trained men working with the precision of a football team. Would such machinery be found satisfactory if used, perhaps, only two or three times a year?

Commercial requirements had evolved a type of machinery designed for dumping large quantities of coal into vessels more or less adapted to receive it as rapidly as it could be delivered from chutes. How could such devices be modified to serve war vessels designed primarily for fighting, and with coal bunkers arranged to fill space not otherwise required, or placed to protect the machinery and magazines rather than with a view of being readily filled by any known mechanical appliance?

While keenly alive to the difficulties to be overcome, the desirability for a successful or even partially successful result seemed to Naval officers worth a serious effort, and accordingly men were detailed to examine and report on all existing forms of coal-handling machinery, with a view to their adaptability to Naval requirements.

While it would doubtless be unjust to give all the credit for instigating this action to any one man, certainly much is due to the official and unofficial efforts of Rear-Admiral Royal B. Bradford, then Chief of the Bureau of Equipment, part of whose duty was to provide coal for the Navy in all parts of the globe. Admiral Bradford not only believed in installing mechanical appliances at those coal depots most frequently visited by our war vessels or likely to be visited frequently by them in time of war, but also continually urged the establishment of new and larger coal depots all over the world, wherever suitable sites could be secured. He thought it

would do no harm if the coaling station were a little more elaborate than perhaps might have answered the purpose, and his motto was: "The only way to have enough coal is to have too much."

Under such auspices, then, a Board was appointed, on November 22d, 1899, consisting of Captain Henry C. Taylor, Captain C. H. Stockton, and Lieut.-Commander Herbert Winslow, to select a site for a coal depot on Narragansett Bay.

At that time the Navy Department had coaling stations at New London, Key West, Pensacola, San Juan and Hawaii. Those at Key West and New London were equipped with elaborate modern appliances.

The establishment of depots was also in progress at Frenchmans Bay, Dry Tortugas, Sitka, Mare Island and Samoa. The stations at Frenchmans Bay and Mare Island were to be fully equipped with the best coal-handling appliances available.

In addition to the foregoing stations, completed or in process of construction, the Bureau of Equipment was seeking to establish stations at Portsmouth, Boston, New York, League Island, Washington, Norfolk, Port Royal, New Orleans, Dutch Harbor, Puget Sound, San Francisco Bay, San Diego, Guam and Cavite. In addition to these, there were to be a number of smaller sub-depots at which, with the possible exception of Olongapo, no great stock of coal was expected to be carried.

Stations at Portsmouth, Boston, New York, Washington, Puget Sound and Cavite have been completed since that time, and stations at New Orleans and Olongapo are now under construction.

The investigations of the Board on Site for the Narragansett Bay Coal Depot led to the purchase of a tract of land containing about 160 acres, on the west side of the Island of Rhode Island, about 7 miles north of Newport, which offered a flat, wide foreshore in the shape of a cape or promontory of solid earth and gravel, providing a firm foundation for any structure which might have to be built. It was bounded on the west by deep water, thus giving access for the largest vessels, without requiring dredging. The normal rise and fall of the tide was only 4 ft. Through the upland part of the tract there flowed a small stream which could be depended on throughout the year for an ample supply of fresh water. All the water and water rights of this stream were secured by the

Government by the purchase of the land through which it flowed. A branch of the New York, New Haven and Hartford Railroad, running from Newport to Boston, passed through the property, and on the flat promontory jutting out into the bay and ending in Coggeshall Point there was a fresh-water lagoon, formed by the previously mentioned stream, covering about 30 acres, which seemed to be inviting for future conversion into a basin for harboring torpedo boats and small auxiliary-service vessels.

The land was fairly fertile, and sufficient was purchased to maintain the horses or other animals which might be found useful at the depot. The spot was healthful and even health-giving. Indeed, the property had belonged to the Government during the Civil War, and had been used for a convalescent camp and hospital. Fig. 4 is a plat of the property.

The strategical importance of Narragansett Bay, for the defense of the Coasts of the New England and Middle States, is well brought out in Captain Mahan's book,* in which he quotes from a private letter from the English Admiral, Sir George B. Rodney (afterward Lord Rodney), to the First Lord of the Admiralty, referring to Clinton's evacuation of Newport:

"The evacuating of Rhode Island was the most fatal measure that could possibly be adopted. It gave up the best and noblest harbor in America, from which squadrons in 48 hours could blockade the three Capital Cities of America—namely, Boston, New York and Philadelphia."

The site selected for the depot, therefore, may be described in one word—"ideal." It remained only to plan a coaling station commensurate with the opportunity, so as to utilize the full resources of the place.

A Board was appointed, on August 7th, 1900, consisting of Captain George A. Converse (now Rear-Admiral, Chief of Bureau of Navigation), Captain C. S. Sperry, and Civil Engineer H. H. Rousseau, to prepare general specifications and estimates for the development of the property. This resulted in the establishment of the Narragansett Bay Coal Depot—the subject of this paper.

It was decided to build a wharf, parallel to the shore, with its outer face in 5 fathoms of water at mean low water, and to con-

* "The Influence of Sea Power upon History."

struct a silo or pocket for suitably housing the coal on shore, where such a structure could be built at less cost than over deep water. The Board anticipated receiving coal ordinarily by water, but recommended utilizing the railroad, also, as an alternate means of supply. It was recommended to commence the depot with a pocket having a capacity of 10 000 tons, arranged so as to be extended readily in the future. Two steam, digging cranes, for mining cargo coal out of vessels, were recommended for the wharf, and cable roads were suggested for carrying the coal from the wharf to the storage pocket on shore and from the latter to the wharf for distribution.

The Bureau of Equipment made an elaborate hydrographic survey of the premises, made borings, drove test-piles, and fixed the size and position of the wharf. It was to be 400 ft. long and 55 ft. wide, and to have its long side practically parallel to the shore and about 500 ft. from it. It was to be connected with the shore at the north end by an earth-filled approach out to the 5-ft. contour, and beyond that with a wooden pile wharf, of somewhat lighter construction than the main wharf, and 48 ft. wide.

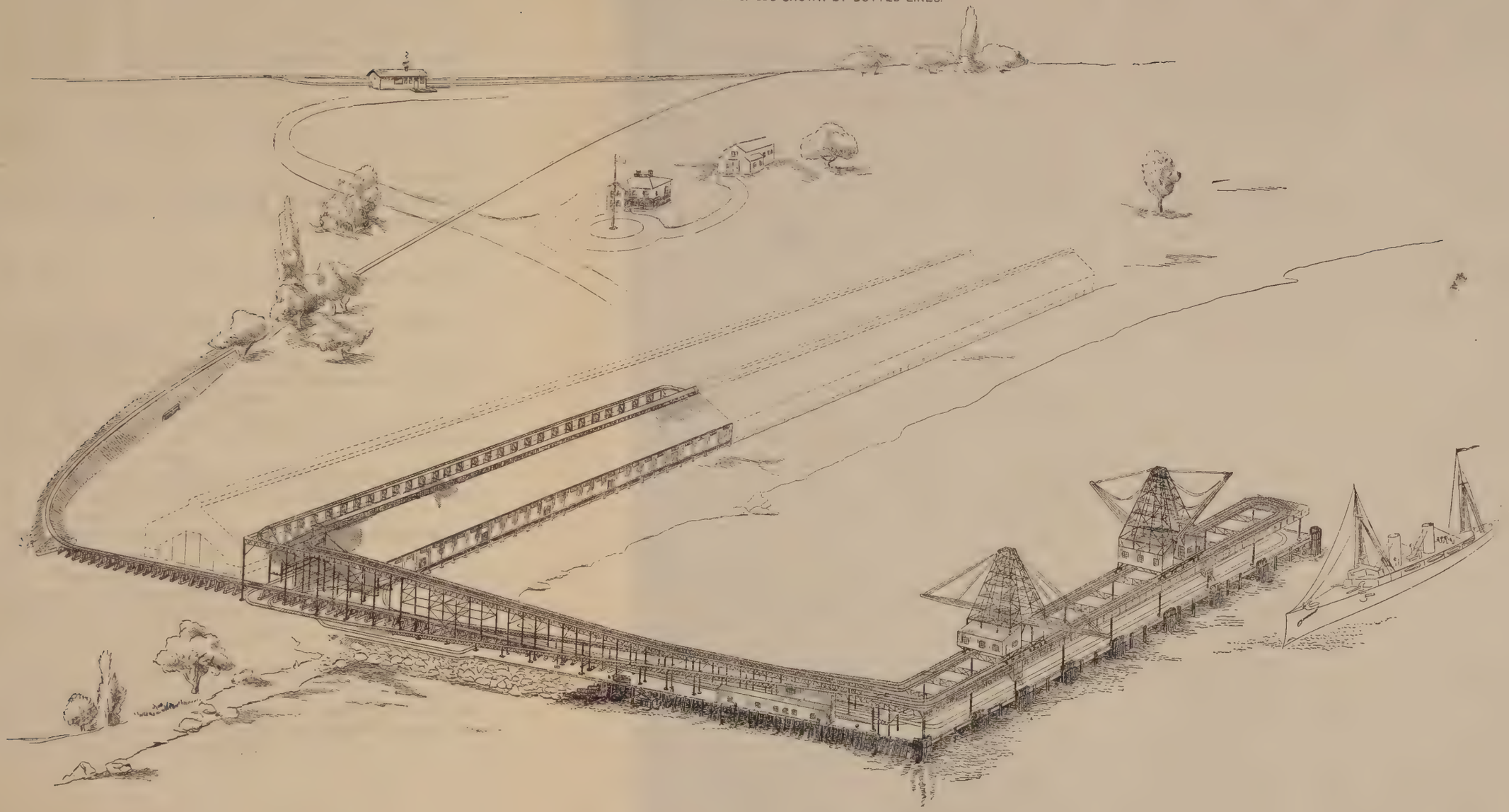
The coal storage building was to be built so as to protect the coal thoroughly from the weather and be provided with gates or valves through which the coal could be withdrawn by gravity into the cable cars. It was to be constructed of metal or concrete, with hollow, cross fire-walls about 75 ft. apart, and equipped with suitable water pipes and hose. In addition to these precautions against fire, the depth to which the coal was to be piled in storage was limited to 15 ft., and thermostats were to be placed at frequent intervals throughout the coal pile and connected with an annunciator and alarm bell, so that any heating of the coal could be quickly detected and located. The cable roads had to be arranged so that, in the event of heating, coal could be readily withdrawn from any part of the pile and replaced at some other point, which would liberate the gas and give the coal a chance to cool.

These elaborate safeguards against fire were thought to be especially necessary for a Naval coal depot, where a pile of bituminous coal might be stored for a long time without being necessarily disturbed.

PLATE XVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVII, No. 1032.
SMITH ON
COALING STATION.

NARRAGANSETT BAY COAL DEPOT

ORIGINAL INSTALLATION SHOWN BY FULL LINES
EXTENSION OF STORAGE SHEDS SHOWN BY DOTTED LINES.



In order to stimulate the interest and co-operation of firms making a specialty of the construction of plants for the storage and handling of coal, as well as to avoid committing the Navy Department to the use of any patented appliance, the specifications prepared by the Board and subsequently issued by the Bureau of Equipment were of a general nature, setting forth the objects to be obtained and outlining the plan which seemed best suited for obtaining them, together with the usual stipulations governing the quality of material and workmanship required.

The subject, having been thus studied in its main features by the Naval authorities, was opened to civil co-operation, and competitive plans and specifications with tenders for construction were invited by public advertisement. The following extract from the specifications outlines the problem briefly as it was presented to prospective bidders:

"The objects to be accomplished by the coal-handling machinery are:

"*First.*—Removing coal from colliers or barges alongside the wharf and delivering into the shed.

"*Second.*—Removing coal from shed and delivery into collier or barge alongside wharf at any point along the outer or inner faces of the L, either in bulk, bags or baskets.

"*Third.*—Removing coal from shed and delivering aboard war vessels along the inner or outer face of L in a convenient manner, either in bulk, bags, or baskets.

"*Fourth.*—Removing coal from railroad cars by hand, or by towers, if possible, and delivering into the shed or on board vessels alongside the wharf.

"*Fifth.*—Removing coal from the shed and delivering into railway cars.

"*Sixth.*—Removing coal from the bottom of the pile in the shed and replacing it in the shed where desired.

"*Seventh.*—Removing coal from a vessel lying on one side of the wharf and delivering it on board a vessel on the other side of the wharf.

"The second and third of the above requirements are considered most important, and it is desired that the plant be equipped with all appliances that will enable all classes of vessels to receive coal rapidly by machinery, supplemented by all possible hand means, and as fast as possible by hand means alone."

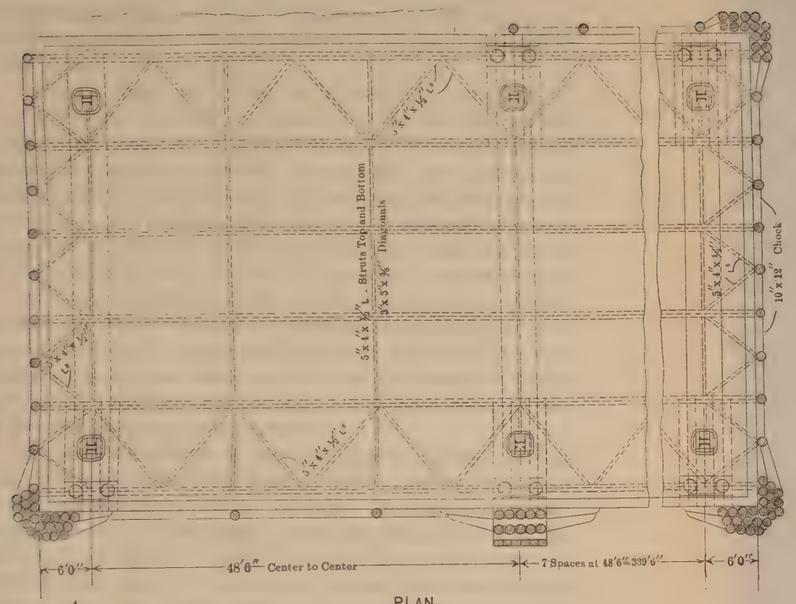
No limit of cost was imposed, though the bidders generally understood that the Bureau of Equipment expected to get a satisfactory installation for less than $500 000.

As was to be expected, even with the very carefully prepared specifications issued, the plans and prices submitted varied greatly. No less than thirteen essentially different plans were received from six bidders, and many of the bidders included various more or less important modifications of their fundamental idea, so that forty-two formal bids or prices were submitted to the Chief of the Bureau of Equipment to consider and make an award.

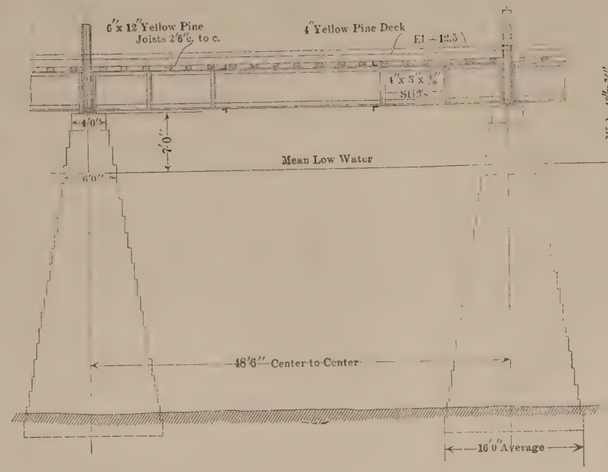
The contract was finally awarded to the writer, on one of his plans, slightly modified to meet the views of the Bureau, for $349 137, and construction was at once commenced. Plate XVIII is an orthographic projection of the design selected. The storage sheds shown in dotted lines were added subsequently under another contract.

A brief description of the plan adopted may be commenced with the main wharf. This consists of nine granite piers, 16 by 55 ft. on the bottom and 4 by 52 ft. on top, finishing at 7 ft. above mean low water, which made the average height of the piers about 35 ft.

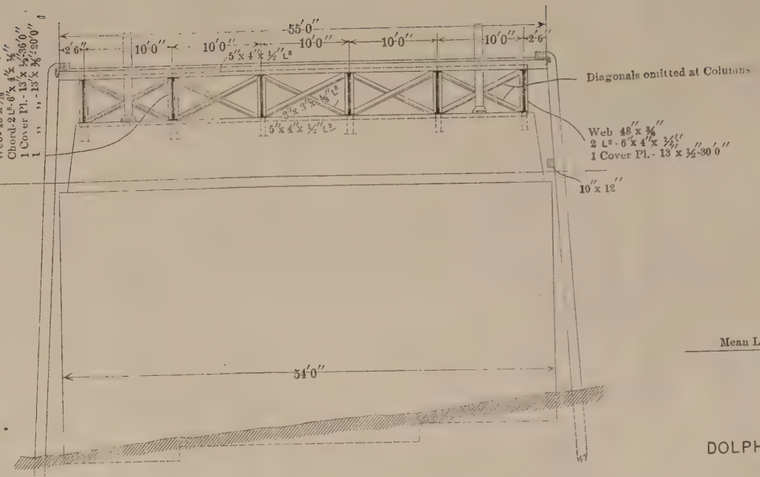
Below mean low water these piers were of granite blocks, averaging about 3 tons in weight, cut to approximately rectangular prisms of about the same shape as bricks, and laid in good bond, with open joints, by a diver. There was no dredging for the footing course except that done by the diver with a hoe. It was expected that the piers would settle about 18 in. into an overlying stratum of sand, which settlement, with the construction adopted, would do no harm if it ceased before the pier above mean low water was laid. The portion of the pier above mean low water was laid with closer joints, in Portland-cement mortar. It should be noted that the portion of each pier below mean low water weighed in water about 900 tons, while the portion above that level weighed about 150 tons. The superimposed weight of the wharf and superstructure resting on each pier was estimated to weigh not more than 100 tons. As soon as each pier reached mean low water level there was placed on it a load of granite blocks weighing considerably more than the total weight of the cement-laid portion of the



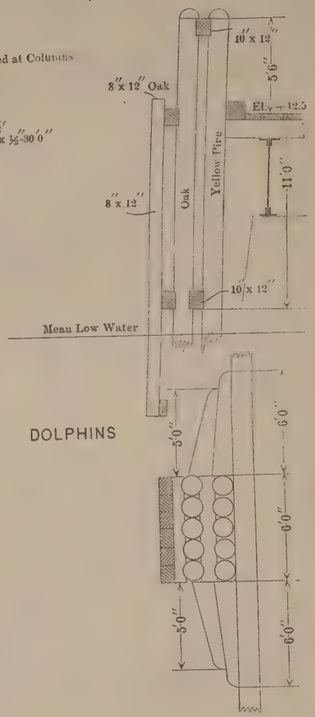
PLAN



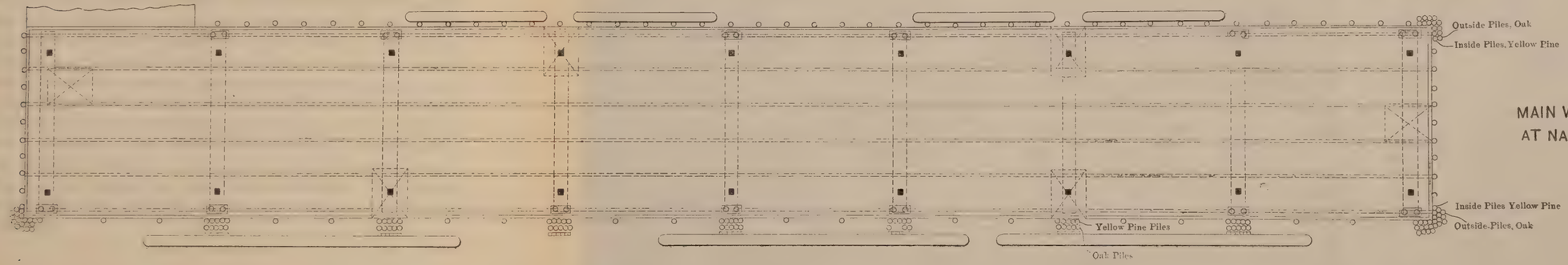
PARTIAL SIDE ELEVATION



SECTION



DOLPHINS



MAIN WHARF FOR U.S. COALING PLANT AT NARRAGANSETT BAY COAL DEPOT.

pier, the wharf, deck and superstructure, which load was allowed to remain until all settlement had ceased. In one sense, it was fortunate that there was great delay in securing the iron superstructure of the wharf, for the settlement of the main piers did not cease as soon as was expected.

Accurate measurements could not very well be made until the top of the pier showed above mean low water, and then accurate readings were taken with a level about once a month until the settlement had stopped entirely for two months. The observed settlement after the piers showed above mean low water was approximately 5 in., and was remarkably uniform. Undoubtedly, the total settlement in the bottom was considerably more than this, as the initial settlement under the 900 tons must have been quite rapid.

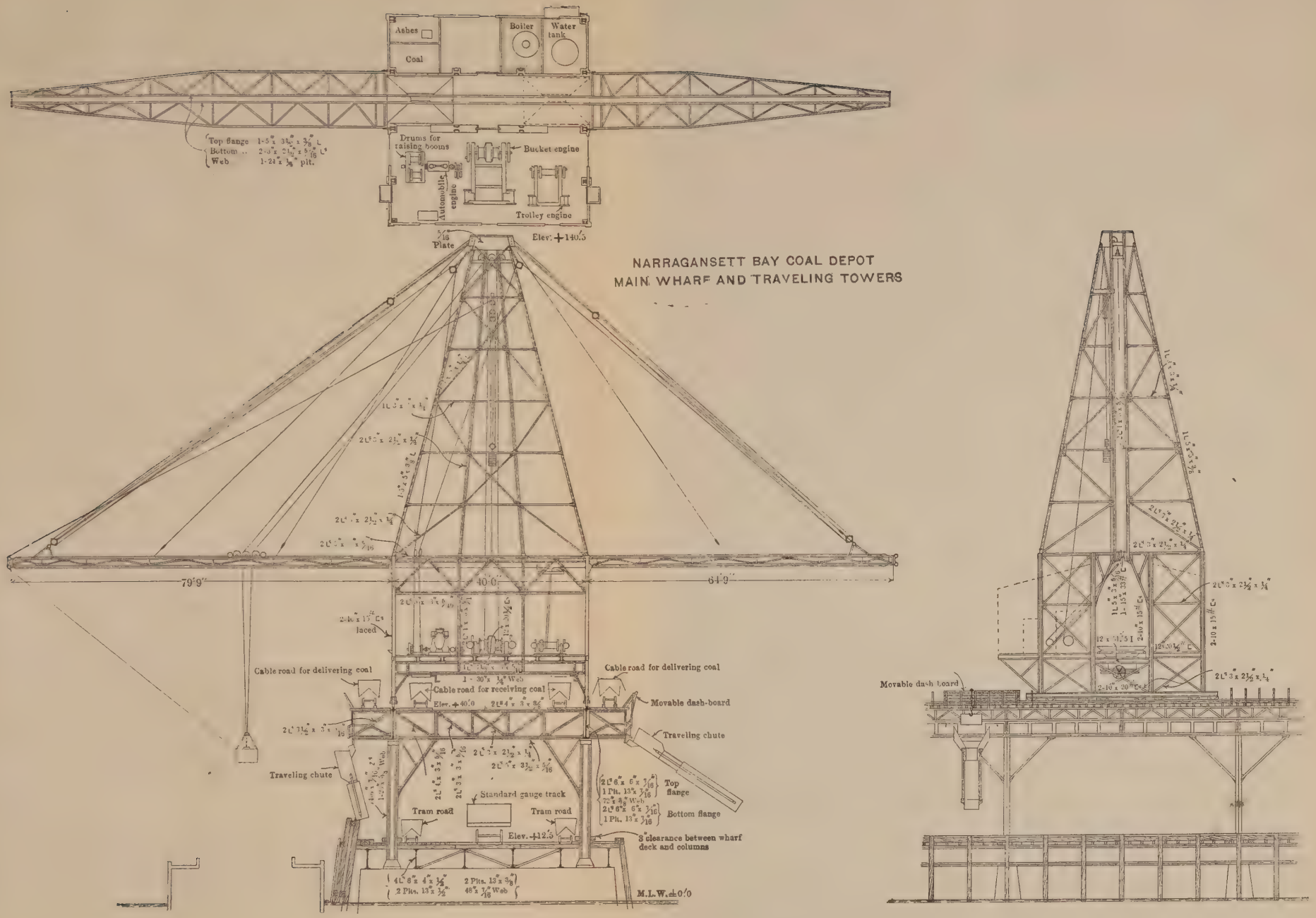
Of course, the primary object in adopting such a questionable method was to save the expense of dredging, and it might be worth noting here that the writer would not favor repeating it for work of similar importance, though, as far as can be detected, no further settlement of the wharf has taken place, and it has now stood true to line and grade since August, 1902.

The piers are 48 ft. 6 in. apart from center to center, and on them six rows of steel-plate girders are laid, designed to carry a distributed dead load of 400 lb. per sq. ft. These girders are thoroughly braced by angles having a minimum thickness of $\frac{3}{8}$ in., so as to resist the shock of a vessel as one solid frame, 50 by 400 ft. in area, weighing nearly 300 tons without the deck, which weighs about 400 tons more. The friction of this weight was relied on mainly for withstanding the impact of ships, and the anchor bolts securing the ironwork to the masonry were made small enough to shear, in the case of excessive strain, without injuring the masonry substructure. The deck and joists are of hard pine, with oak fenders and fender piles. A somewhat ingenious fender pile construction was devised by the Bureau, and is shown in detail on Plate XIX. The prototype of this was found in the fenders at the pier of the Fall River Line, in Newport. By this device the resiliency of two sets of piles was utilized in absorbing the momentum of an approaching vessel. Thus far, the fenders have been very satisfactory. The general construction of the wharf will be understood better by reference to Plate XIX.

Starting again from the piers, and quite independent of the steel and woodwork of the wharf, there are 18 steel columns, 2 on each pier, to carry the viaduct on which the unloading towers travel. These columns also support the wharf loops of the cable road. The cross-section of this structure is shown on Plate XX. It was carried to a height of 40 ft. 6 in. above mean low water and no higher, in order to keep the grade of the cable road (used in bringing coal out from under the storage pocket, to the top of the trestle) from exceeding 7%, which was specified as a maximum by the Bureau of Equipment. Experience has proved that if the delivery track on the wharf trestle had been 10 ft. higher, it would have been more convenient for delivering coal to vessels having a high freeboard.

Two cranes or towers were provided for mining coal out of vessels, in which it is generally received at the depot. One of these cranes is shown on Plate XX. They are equipped with outshore booms having an effective reach of 60 ft. from the outer face of the wharf, and inshore booms having an effective reach of 45 ft. from the inner face. These booms are arranged to fold up in order to clear the masts of vessels lying at the wharf. A 1-ton clam-shell bucket is operated by friction drums mounted directly on the crank shaft of a hoisting engine having 14 by 24-in. double cylinders. The two wire cables, to open and close the bucket, pass over sheaves in a trolley free to move horizontally on the boom and thence over sheaves in the top of the tower to the main winding drums. The weight of the bucket is sufficient to bring the trolley home toward the center of the tower, and a separate engine having 10 by 15-in. double cylinders hauls the trolley out toward the end of either boom with a continuous rope passing over sheaves in the end of each boom.

This type of machinery has been perfected, and was installed by the Mead-Morrison Manufacturing Company, of Boston and New York. It is known as direct-connected machinery of the two-men type. The bucket is handled by two independent operators, one of whom moves the trolley while the other opens, closes and raises the bucket. After some practice, the men become wonderfully expert, and work in such harmony that the bucket describes a curve resembling a parabola—one operator arranging the ordinates while the other adjusts the abscissas—and it has been demon-


Ashes
Coal
Boiler
Water tank
Drums for raising booms
Bucket engine
Automobile engine
Trolley engine
Elev. +140.5
Plate
Top flange 1-5" x 3½" x 3/8" L
Bottom .. 2-5" x 2½" x 5/16" L
Web 1-24" x 3/8" plt.
NARRAGANSETT BAY COAL DEPOT
MAIN WHARF AND TRAVELING TOWERS
79'9"
40'0"
64'9"
2-10" x 15" C's laced
Cable road for delivering coal
Cable road for receiving coal
Elev. +40.0
1 - 30" x ¼" Web
Movable dash-board
Traveling chute
Standard gauge track
Tram road
Tram road
Elev. +12.5
Top flange
Bottom flange
3' clearance between wharf deck and columns
M.L.W. ±0.0
Movable dash board

strated that the expense of the extra man is more than repaid by the greater speed attained. These towers have been worked for a short time at the rate of nearly 200 tons per hour for each tower, and the men have no difficulty in maintaining a rate of 100 tons per hour as long as there is coal within reach of the bucket.

The low trestle, previously referred to as a defect in discharging coal from the station, has an offsetting and obvious advantage in taking coal into the station. It might be said, however, that such high rates of speed in taking coal into storage are of doubtful utility in the case of a Naval coaling station. The rate required by the Bureau of Equipment in the case of this installation was only 50 tons per hour for each tower.

The booms on these towers are raised and lowered and the towers propelled along the viaduct by a separate engine having two 7 by 8-in. cylinders. A 100-h. p. vertical boiler, supplying steam at 80 lb. pressure, with a large water-tank and coal-bin, was set up in a boiler-room entirely separate from the engine-room, as shown by the drawing. The isolation of the boiler-room from the engine-room has proved to be not altogether advantageous, though it was almost a necessity in towers of the type adopted. The towers are estimated to weigh about 135 tons each. They travel on a track of 40-ft. gauge from center to center of rails.

The object of having the long outshore boom was to handle coal in tubs brought on platform cable cars for delivery to any point on the deck of a battleship lying alongside. This tremendous reach fixed the height of the tower, and was a factor in fixing the base in order to obtain the necessary rigidity.

Thus far, the operation of lifting loaded tubs of coal across the decks of naval vessels has not been looked upon favorably by captains coming for coal, and if this function of the tower is not to be utilized, it is regrettable that the long boom was provided, thus making the towers more heavy and unwieldy than was strictly necessary.

It will be seen that the towers, as built, comply with all the requirements of the Bureau of Equipment. They can be used for unloading coal from vessels lying at either side of the wharf; for lifting coal in tubs off platform cars delivered by the cable road to

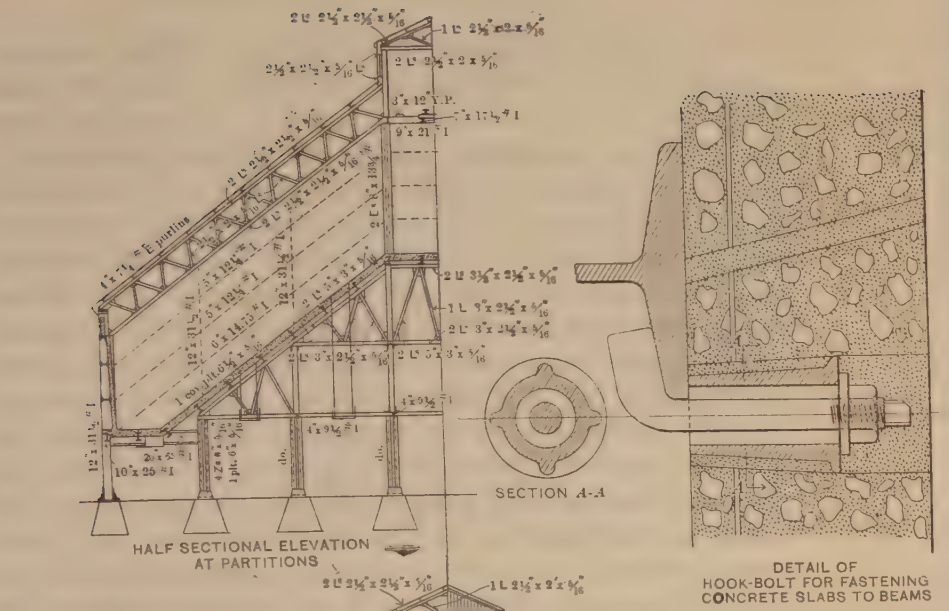
serve vessels lying on either side of the wharf; for transferring coal from a vessel lying on one side of the wharf to a vessel lying on the other side; for mining coal out of railroad cars switched to a standard-gauge track, carried down the center of the wharf on the main deck, as shown on Plate XX, which coal could be deposited either in a hopper for transmission by the cable road into the storage building or directly to a vessel lying on either side of the wharf. In addition to this, the booms have been designed with sufficient strength to permit the towers to be used as cranes in an emergency, as they have a lifting capacity of 20 tons for a limited reach.

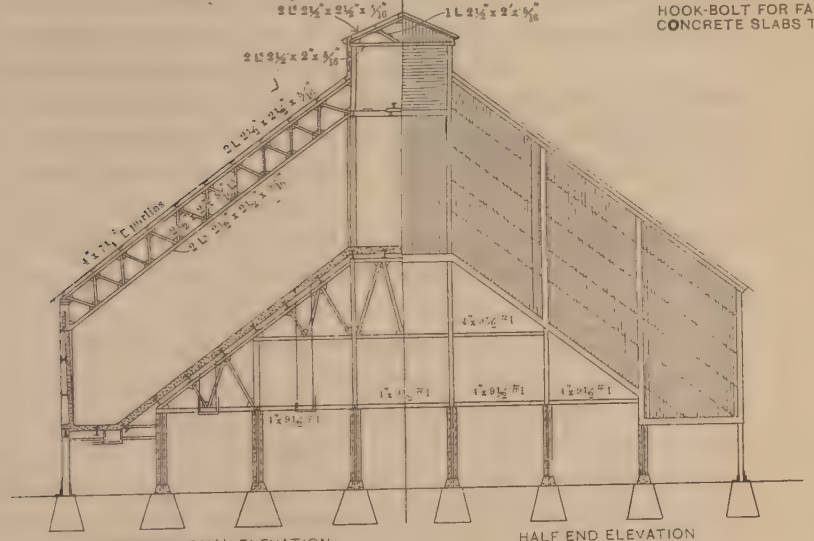
The hopper in the tower, intended to receive coal from the clam-shell bucket and transfer it to the cable cars which take it into the storage building, was built on a track so that it could be shifted from one side of the tower to the other, as indicated by the dotted lines. This was done so that the hopper could be placed as near as possible to the boat to be unloaded, and thus save travel for the clam-shell bucket and a reversal of strain in the cable controlling the trolley.

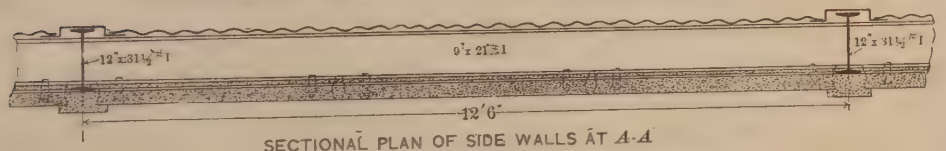
From the hopper in the towers, the coal is taken away on a cable road equipped with steel cars, having a capacity of 2 tons each, and arranged to discharge automatically, by side-opening doors, released by a trip set in the track at any desired point. The cable track is carried by a steel trestle, rising at a 7% grade to the top of the storage building, which was constructed on shore at some distance from the main wharf, for reasons previously stated.

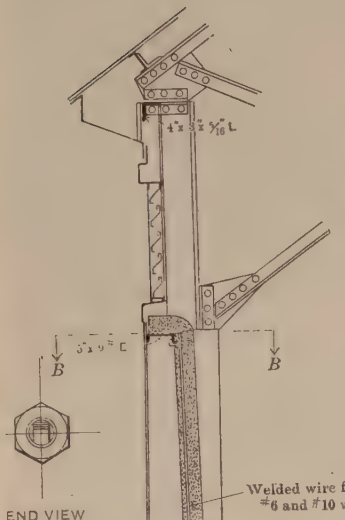
The construction of the storage building is shown by Fig. 1. Concrete piers were carried below the frost line and spread out so that the pressure on the soil was reduced to a little more than 2 tons per sq. ft. On these piers a steel frame, carrying ferro-concrete floors and walls, and roofed with galvanized, corrugated iron, forms the coal storage pocket. Owing to the slope of the floor, 9 in 12, the depth of the coal, while 15 ft. measured vertically, is only 12 ft. when measured normal to the surface. Notwithstanding this shallow depth, one consignment of coal has already shown a dangerous tendency toward heating.

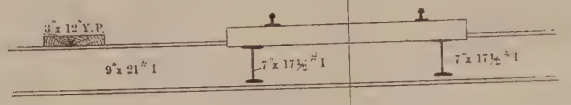
Galvanized corrugated-iron walls, with pilasters covering the steel posts of the building, were built around the sides and ends of

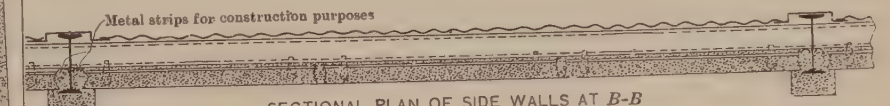


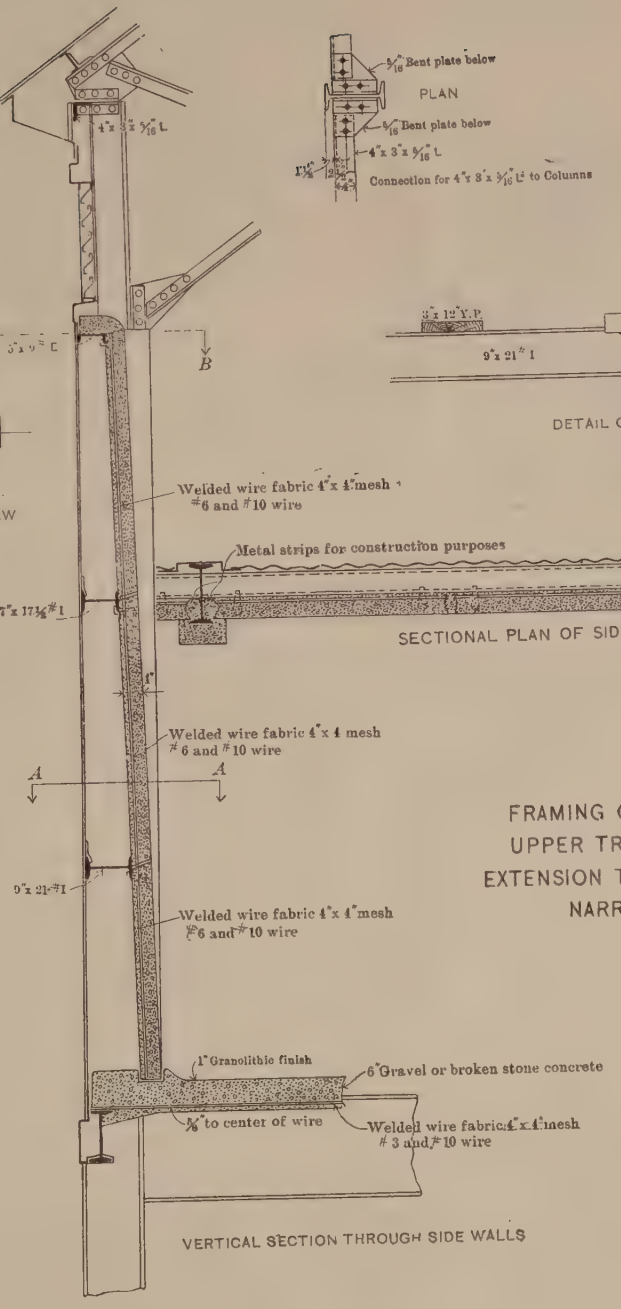











Welded wire fabric 4"x 4" mesh
#6 and #10 wire
Metal strips for construction purposes
Welded wire fabric 4"x 4 mesh
#6 and #10 wire
Welded wire fabric 4"x 4" mesh
#6 and #10 wire
1" Granolithic finish
6" Gravel or broken stone concrete
5/8" to center of wire
Welded wire fabric 4"x 4" mesh
#3 and #10 wire
VERTICAL SECTION THROUGH SIDE WALLS

FRAMING OF SIDE AND END WALLS, UPPER TRACKS AND WALKWAYS OF EXTENSION TO U.S. NAVAL COAL DEPOT NARRAGANSETT BAY, R. I

the pocket, partly for the sake of the appearance, and partly, perhaps, with a view of protecting the thin concrete walls from the action of frost.

The latter precaution was probably unnecessary, and in future construction of a similar kind it might be advisable to save the cost of the outer sheet-metal covering. In the case of this building, the double walls involved some difficulties in construction. It was decided to put up the corrugated iron first so as to secure it thoroughly to the steel framework of the pocket, and protect subsequent construction work from the weather. It was then found to be impracticable to withdraw the outer centering of the concrete wall, and this had to be left in the space between the concrete wall and the outer sheet-metal covering. If the alternate method of building the concrete wall first had been adopted, the sheet-metal covering would have had to be secured by nails to wooden nailing strips, as there was not sufficient space between the walls to permit of riveting. In either case, some wood had to be left in the space between the walls of a supposedly fire-proof storage shed.

In an extension to this depot, subsequently contracted for by the writer, and consisting of three additional storage buildings equal in size and similar in construction to the first, this difficulty was overcome in the following way:

The walls and partitions of the building were made of cast concrete slabs, 4 in. thick and about 4 ft. square, in which a reinforcement of Clinton wire-cloth had been inserted. After these slabs had hardened from 2 to 6 weeks, according to the time of year, they were lifted into the building and secured to the steel framework by special hook-bolts, very much as slabs of slate might be set. This method of construction insured a much stronger wall than in the original pocket, as each slab could be thoroughly inspected, and the handling incident to transferring the slab from the place of manufacture to its final place in the wall served as an effectual test of each and every piece. The cost of the construction was also found to be in favor of the cast-slab method.

Plate XXI shows the construction of the extension. The coaling station now has a capacity of 40 000 tons above the inclined floor, subject to a gravity discharge, and additional space for about 20 000 tons under the building, without piling the coal more than 12 ft.

deep. At the time the extension was made, the floors under the building were paved with concrete.

The central columns projecting through the coal above the inclined floor, are protected by concrete in metal cylinders of about 11 in. outside diameter. The columns next to the ground are protected by similar concrete coverings in metal cylinders 15 in. in diameter. Hot coal, therefore, could do little damage to the steel frame of the building and the steel is not exposed to the corrosive action of the coal at any point.

In the outer row of protected columns under the building grooves are cast in the concrete covering for the insertion of flash-boards, to assist in confining the coal in the auxiliary storage space on the ground.

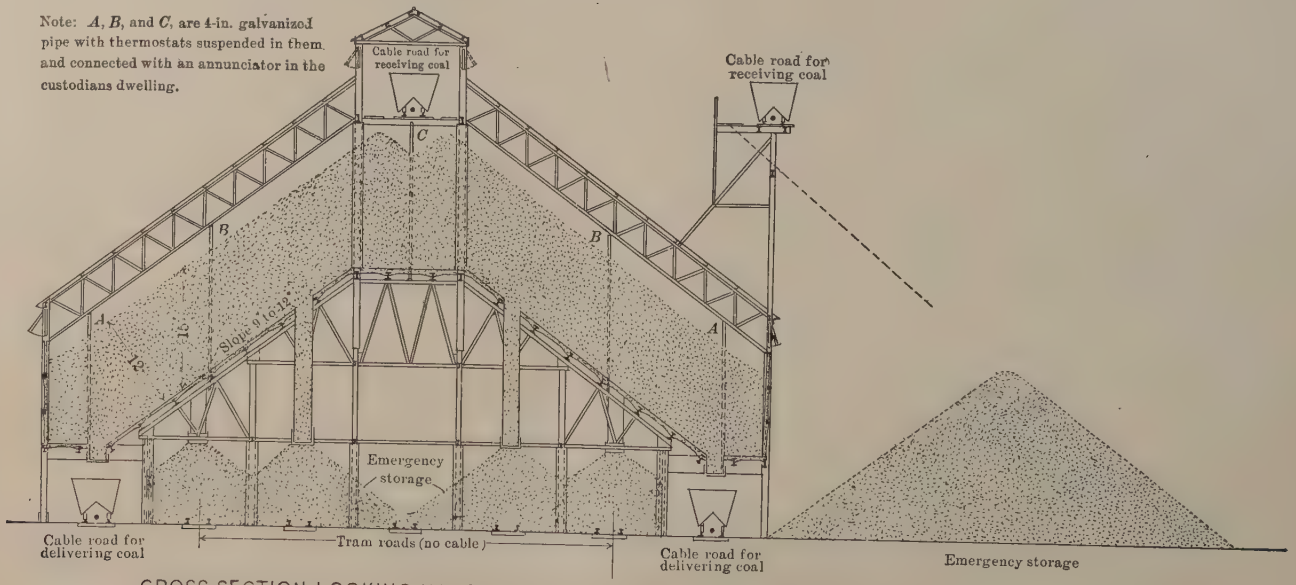
In the original building, the cable road taking the coal into the building was returned over a trestle carried by a continuation of the east line of columns through the roof, which was also intended to serve for forming a large auxiliary pile of coal on the ground east of the shed in an emergency. (Fig. 1).

When the storage building was extended, this trestle was taken down and the cable track returned through the monitor of the new east shed.

A separate cable loop had to be provided on the ground to withdraw coal from the east shed, and the tracks were arranged so that cars could be switched from either shed to either cable. The tracks shown under the middle spaces of the storage buildings are tram tracks on which cars have to be backed by hand, in case the auxiliary storage under the building is utilized. These cars would be pushed out to the cable track at either end of the building and be moved to the wharf by the cable.

One of the problems was to find for the bottom of the pocket a gate or valve which could be operated with ease and certainty by one man, so as to fill the cable cars without spilling coal on the track, and do this rapidly enough to dispatch a loaded car every 30 seconds.

An overhead gate was sought, in order to save headroom, and the difficulty of handling large lumps of bituminous coal was finally overcome by adopting the gate shown by Fig. 2. It will be seen that the jaws of this gate do not meet, so that there is no chance for



CROSS-SECTION LOOKING NORTH THROUGH STORAGE BUILDING, NARRAGANSETT BAY COAL DEPOT

FIG. 1.

a lump to get between them and prevent their closing, nor does the gate have to be geared with sufficient strength to bite through a lump of coal, which would necessarily have required a very slow movement.

The edges of the valves of this gate miss each other by about 9 in., and there is no trouble in handling lumps of bituminous coal after the men have learned the tricks of the gate.

Each valve of the gate is worked by a separate lever—one controlled by the left hand and the other by the right hand of the operator. To draw coal, Valve *A* is opened before Valve *B*. To shut off, *B* is closed before *A*. The crank and connecting rod controlling *A* pass over a dead center in both positions, so that the valve may be held automatically either open or shut. Valve *B* remains either open or shut by virtue of its weight. The gate works easily, and the head of coal on it appears to make no difference, except that *B* cannot be opened until *A* is opened and the coal commences to flow, when the pressure on *B* seems to be entirely released. The levers are removable, so that the gates cannot be opened by mischievous persons unless equipped with proper tools. This gate is believed to be new.

Since the station has been turned over to the Government, its operation, in the main, has been satisfactory, except perhaps that the towers are rather unwieldy, and the delivery trestle on the wharf might have been higher, as mentioned previously.

The trestle, however, seems to be high enough for loading the colliers and barges for which it was designed, and on a recent occasion the Collier *Caesar,* detailed to assist in towing the dry dock *Dewey* to Manila, was loaded with 2 869 tons of coal in 15 hours, and sailed from the depot 25 hours after her arrival.

The station has been in continuous service since about May 1st, 1904, and the repairs have been inconsiderable. The most serious break was in the cable driver shown by Fig. 3. In July, 1905, both the drums were found to be cracked at the points, *CC,* where the spokes were attached to the rims. The cable road at the time was working under a very moderate load. The cause of these cracks was not at first apparent. Shrinking strains; crystallization; unequal wear of the grooves, causing the winding of the cable to set up an excessive pull between the shafts, on the principle of the differential pulley block; careless or malicious blows of a sledge, were

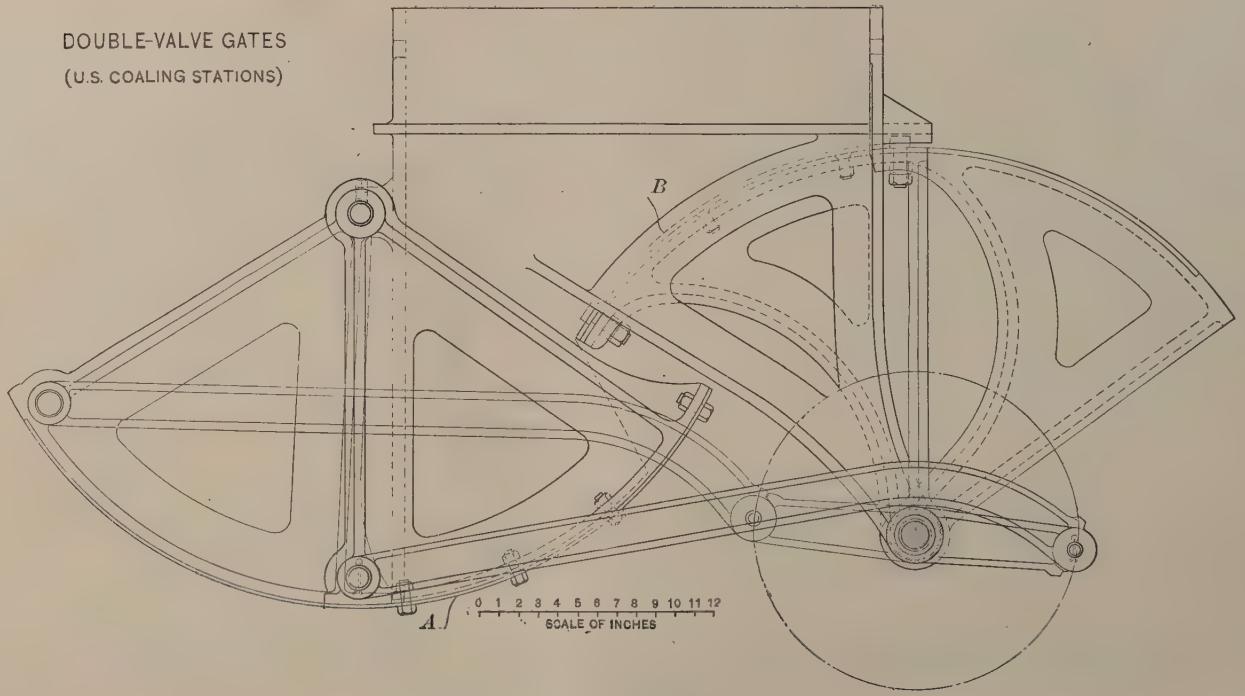
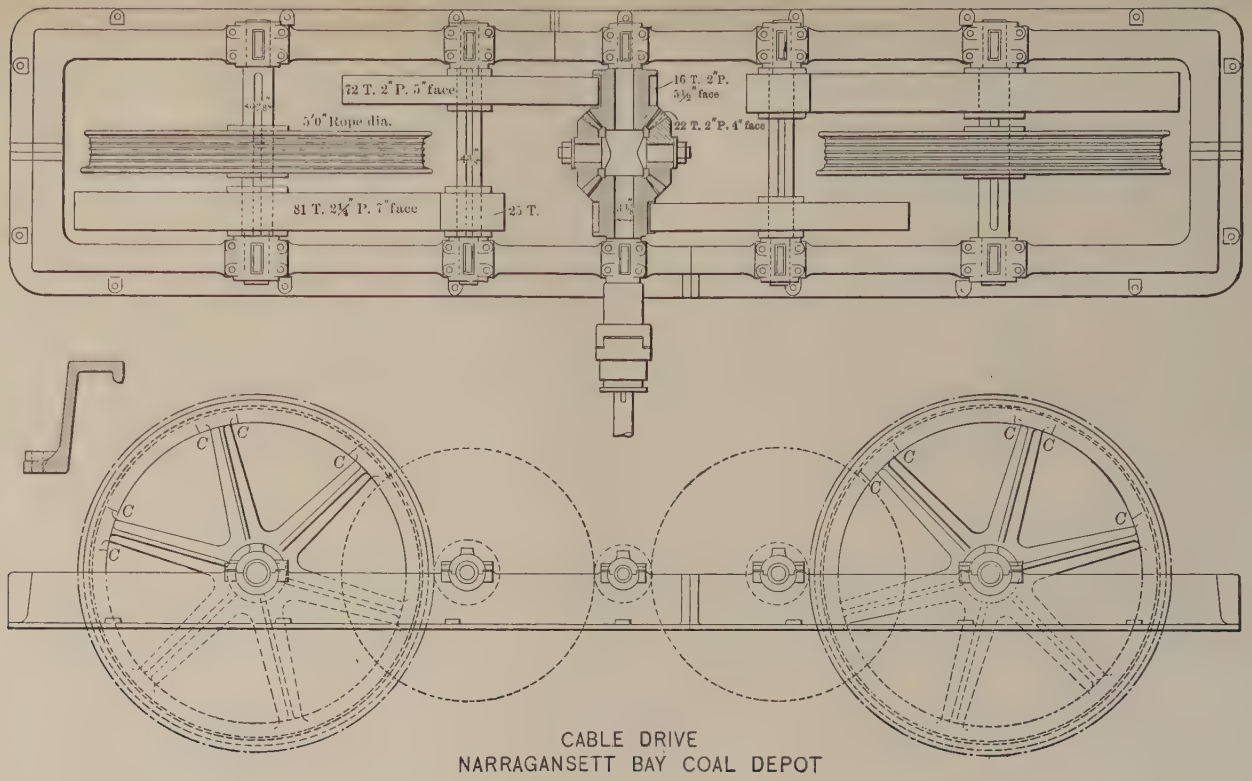

DOUBLE-VALVE GATES
(U.S. COALING STATIONS)
B
A
0 1 2 3 4 5 6 7 8 9 10 11 12
SCALE OF INCHES

Fig. 2.



CABLE DRIVE
NARRAGANSETT BAY COAL DEPOT

FIG. 3.

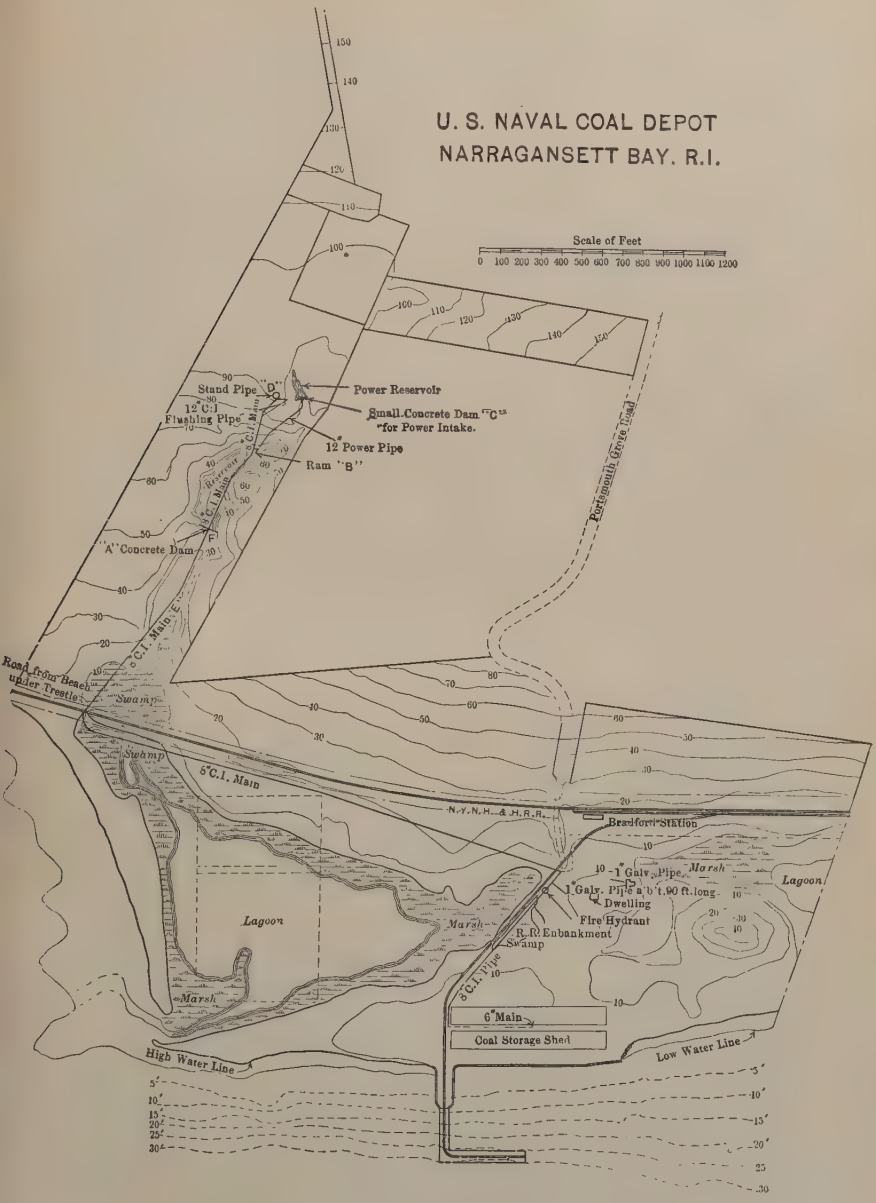

U. S. NAVAL COAL DEPOT
NARRAGANSETT BAY. R.I.
Scale of Feet
0 100 200 300 400 500 600 700 800 900 1000 1100 1200
Stand Pipe "D"
12" C.I. Flushing Pipe
Power Reservoir
Small Concrete Dam "C" for Power Intake.
12" Power Pipe
Ram "B"
"A" Concrete Dam
Portsmouth Grove Road
Road from Beach under Trestle
Swamp
Swamp
5" C.I. Main
N.Y.N.H. & H.R.R.
Bradford Station
1 Galv. Pipe
1 Galv. Pipe a b't 90 ft. long
Dwelling
Fire Hydrant
R.R. Embankment
Swamp
Marsh
Lagoon
Marsh
Lagoon
3" C.I. Pipe
6" Main
Coal Storage Shed
High Water Line
Low Water Line

FIG. 4.

all considered and set aside. It was finally decided that the cracking was due to the key forcing the drum out of concentricity with the shaft to the amount that it was bored larger than the shaft—namely, about $\frac{1}{64}$ in., so that the drums revolved as cams instead of as true cylinders. The differential gearing allowed one drum to advance or retreat angularly with respect to the other, so that it was quite possible that one cam would be frequently in opposition to the other, causing four parts of $\frac{7}{8}$-in. steel rope to put too great a strain on the arched rim of the six-spoke drum.

A description of the Narragansett Bay Coal Depot would be incomplete without mentioning the water-works installed by the Bureau of Equipment, though the writer had nothing to do with this part of the work.

Fig. 4 shows the general location and arrangement of the water-works system. *A* is a concrete dam, impounding about 3 000 000 gal. of water, the top of the spillway being at Elevation 40 above mean low water. *B* is a duplex hydraulic ram driven by a 12-in. power pipe running from the dam at *C*, which impounds a comparatively small volume of water at Elevation + 81. *D* is a 275 000-gal. stand-pipe, filled by an 8-in. pipe from the ram. The remainder of the water wastes into the lower reservoir. The bottom of the stand-pipe is at Elevation 93 above mean low water. It is 30 ft. in diameter and 52 ft. high. *E* is an 8-in. cast-iron pipe, supplying the fire hydrants and water supply system at the station.

By suitable valves at *F*, water can be drawn either from the stand-pipe under the full head, or from the main reservoir under a head of about 40 ft. The lower head is ample for supplying water to ships and for similar purposes, but, on account of the fire risk, the full pressure of the stand-pipe is usually carried in the main. This pressure is sufficient to throw a stream from a $2\frac{1}{2}$-in. hose over one of the storage buildings.

It is obvious that a fleet of several vessels could not be coaled satisfactorily by lying alongside the wharf at such a station as has been described, even if the captains had no objection to having huge tubs of coal dangling over the decks, for the reason that only one vessel could lie alongside the wharf at one time, and it would be necessary to allow at least one day for coaling each ship.

For fleet or squadron service, the Bureau of Equipment has provided twelve barges, which can be readily loaded from the tres-

tle, and, as a matter of fact, these are kept loaded ready to be taken alongside a war ship needing coal at a few minutes' notice. With such an auxiliary fleet of barges, the ships of a squadron could lie at their moorings, without coming alongside the station, and take coal from the barges quite as rapidly as if they were immediately under the coaling appliances. More manual labor is required with the barge system, but there are usually men enough on Naval vessels to minimize this disadvantage.

Water barges and a tug have recently been added to the auxiliary fleet, and to protect these the Bureau has since constructed a granite breakwater, shown by Fig. 5, on substantially the same general specifications as adopted for the nine piers of the main wharf, except that it was decided to dredge for the footing course of the breakwater, in order to avoid any possibility of unequal settlement, and eliminate settlement altogether, if possible.

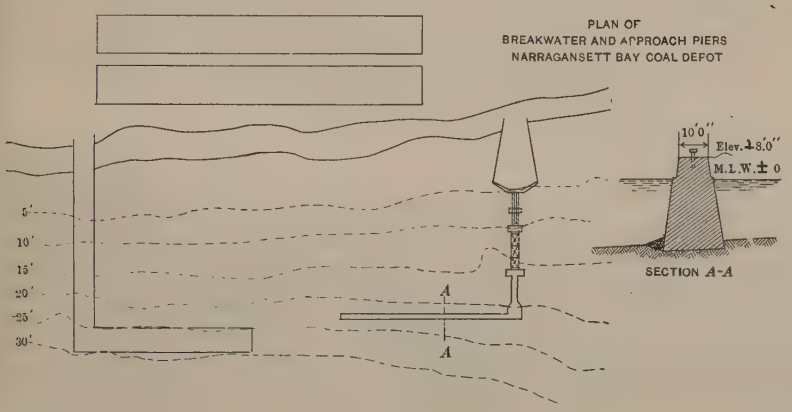


FIG. 5.

The custodian of the depot is a trained man of the Navy, having the rank of Boatswain. The other employees are civilians. The usual force is about 15 men, some of whom are working as laborers in improving the grounds.

Comfortable dwellings, with running water and modern plumbing, are provided by the Government on the reservation for the custodian and the more important men in his force.

An electric-light plant is now being installed, so that the station will soon be available for night as well as for day work.

DISCUSSION.

Mr. Foster. E. H. Foster, M. Am. Soc. C. E.—Mr. Smith has given a most interesting description of a work which is remarkable for its originalty of design. A great many of the details presented problems which called for entirely new solutions bordering closely on inven-

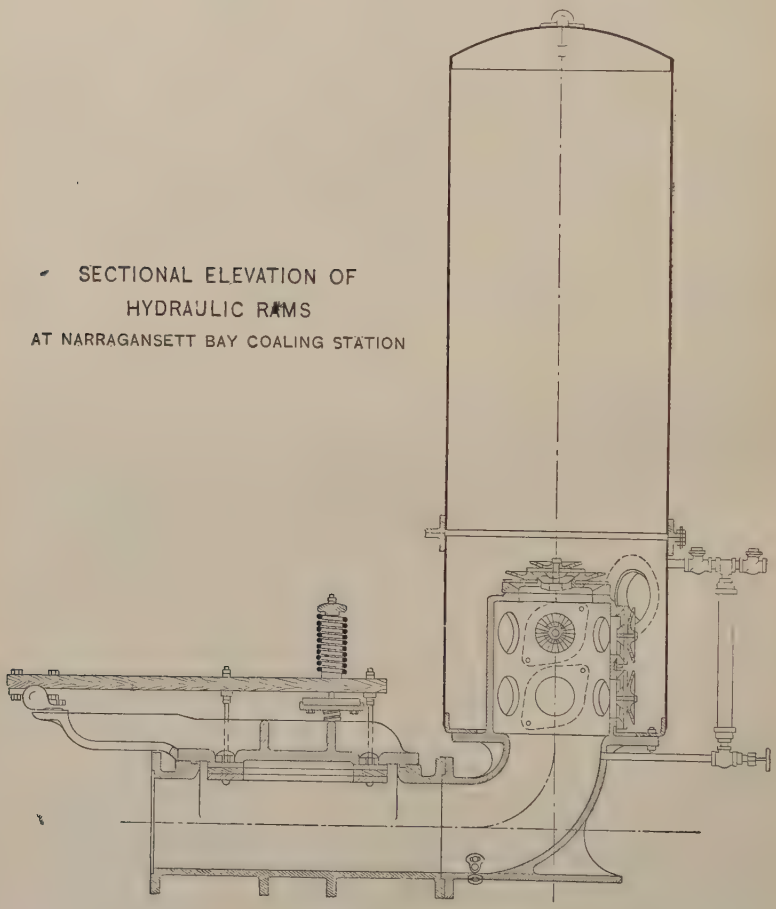


Fig. 6.

tions. In spite of this, however, the plant has been most conspicuously successful from the first time it was put into service; and reflects credit equally upon the designers and contractors.

One of the uncommon features of this plant is the use of hydraulic rams for the water supply. The problem presented was

to keep the stand-pipe full by the power of the water falling from the upper to the lower reservoir. This stream of water, while of ample volume at certain seasons of the year, would shrink to such small proportions, during dry weather, as to be scarcely enough to furnish the necessary power to drive a water-wheel and pump; therefore, it became necessary to seek a more efficient means of utilizing this power. It was desired not to keep an attendant at the pump-house, and, after much consideration, it was decided to install a duplex hydraulic ram with two 12-in. drive pipes. The rams would have been duplex if a common delivery valve chamber had been used; in reality, the delivery valve chambers were made in

Mr. Foster.

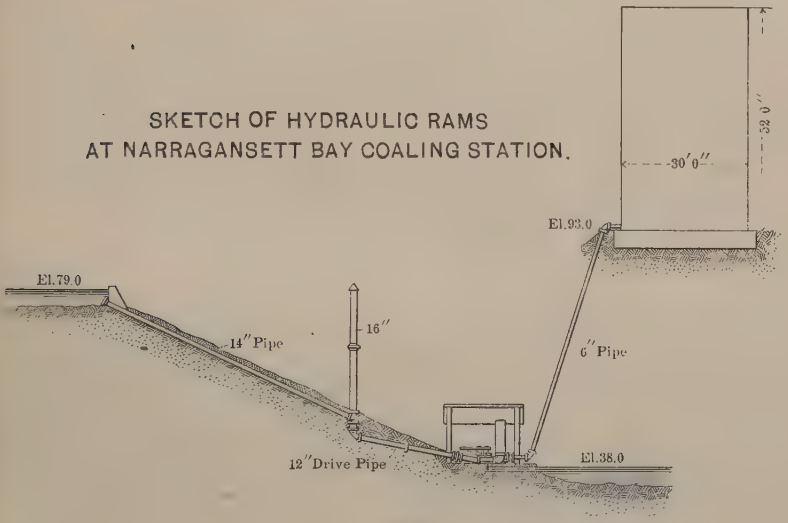


FIG. 7.

two separate units, hence the ram is not strictly duplex, but consists of two independent machines, each exactly like the other, and may be operated separately or together. The conditions under which the rams were installed were that the efficiency should be 70%; also, that they should be capable of filling the stand-pipe in 36 hours when the power water was supplied at the rate of 1 cu. ft. per sec., or in 12 hours with 3 cu. ft. per sec.

The results obtained in the test exceeded all the guaranties considerably, and, in fact, were quite remarkable; they laid bare the hitherto undeveloped possibilities of this kind of machine for pumping water, where water-power exists, without attendance and with high efficiency. These rams were designed specially for this work,

Mr. Foster. and, while perhaps not the largest that have ever been built, were probably the first large rams to be used with such a high working head. It would appear, from the results which these have given, that rams of much larger size could be built with safety.

EXTRACT FROM OFFICIAL TEST OF RAMS.

	No. 1.	No. 2.
Total water delivered to engine, in gallons per minute (Q)	582	578
Water delivered to stand-pipe, in gallons per minute (q)	232	228
Power head, in engines, in feet (H)	36.75	37.25
Pumping head, in feet (h)	47.25	46.85
Strokes per minute	130	130
Efficiency, per cent. (D'Aubuisson formula)	91.11	88.95
Efficiency, per cent. (Rankine formula)	85.22	81.76

In connection with the test of the rams, an interesting discussion arose as to the proper formula to be used in determining the efficiency developed. Two well-known authorities, Rankine and D'Aubuisson, each give formulas which differ materially, thus:

$$\textit{Rankine Formula} \qquad E = \frac{q\,h}{(Q - q)\,H}.$$

$$\textit{D'Aubuisson Formula} \quad E = \frac{q\,(H + h)}{Q\,H}.$$

Under these conditions of fall and pumping head, the D'Aubuisson formula gives an efficiency nearly 7% greater than that of Rankine.

After more or less correspondence, the Department finally ruled that the D'Aubuisson formula was the more logical, and therefore should be accepted as correct, on the grounds that only the rams were actually under test, and that they should be charged with the energy received at the point of reception and credited with the energy delivered at the corresponding point of delivery. In whatever way this matter is regarded, the efficiency is certainly very high, and cannot be approached by any other form of hydraulic motor known to the speaker. It certainly would seem that a very large field awaits the further development of hydraulic rams.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1033.

THE VENTILATION OF TUNNELS.*

By Charles S. Churchill, M. Am. Soc. C. E.

With Discussion by Messrs. George S. Rice, R. P. Bolton and Charles S. Churchill.

In 1904 the writer presented to the International Engineering Congress, Paper No. 43, on the "Ventilation of Tunnels."† Since that date improvements have been made in some of the tunnels and subways referred to, which are worthy of note, and the writer has obtained certain additional data, as to the condition of the air in some subways and tunnels in Europe, which are valuable for comparison in determining good practice.

Gallitzin Tunnel.—The ventilating plant at Gallitzin Tunnel, west of Altoona, on the main line of the Pennsylvania Railroad, was referred to in that discussion as being of the same type as that of the Elkhorn Tunnel, on the Norfolk and Western Railway, and as being practically completed at the date of the discussion.

This ventilated tunnel is 3 600 ft. long, and is built for a single track, being 17 ft. wide and 20 ft. 5 in. high, and is lined with concrete. Its sectional area is 324 sq. ft., and the cubical contents of

* Presented at the meeting of September 19th, 1906.

† Published, with articles by other writers on this subject, in *Transactions*, Am. Soc. C. E., Volume LIV, Part C, pp. 523–579.

the whole tunnel are 1 166 400 cu. ft. There is an ascending grade of 0.5% from the east end through the tunnel, but the east approach is on a heavier grade, so that west-bound freight trains generally have a pusher engine at the rear of each in passing through the tunnel.

Plate XXII shows the general arrangement of the ventilating plant and all important details, its location being at the east or lower end of the tunnel.

With fans making 145 rev. per min. the observed flow of fresh air through this tunnel from east to west is at a velocity of 1 780 lin. ft. per min., equivalent to 576 000 cu. ft. per min. As usually operated, these fans make 130 rev. per min. while freight trains are in the tunnel, and the velocity of the air in the tunnel is 1 550 ft. per min., which results in 502 000 cu. ft. of fresh air being delivered through the tunnel each minute. With the fans running at this speed, the tunnel is cleared directly after the exit of a west-bound train. Excepting when freight trains are in the tunnel, the fans run at from 40 to 60 rev. per min.

There are, as shown by Plate XXII, two 8 by 16-ft. fans, one on each side of the track, each operated by a direct-coupled engine of about 200 h. p., the total rated horse power of the plant being 400. This ventilating device was put into regular operation in April, 1905.

St. Clair Tunnel.—The St. Clair Tunnel, on the Grand Trunk Railway, with its insufficient ventilation, is described in the *Transactions.**

On October 9th, 1904, six employees of the railway were killed in this tunnel by gas and smoke from a locomotive while necessarily standing in the tunnel on account of a break-in-two of a train. This accident is fully described in *The Railway and Engineering Review.*†

In lieu of installing a system of ventilation, the Tunnel Company has during this year contracted for the electrical operation of this tunnel by methods described in detail in *Engineering News.*‡

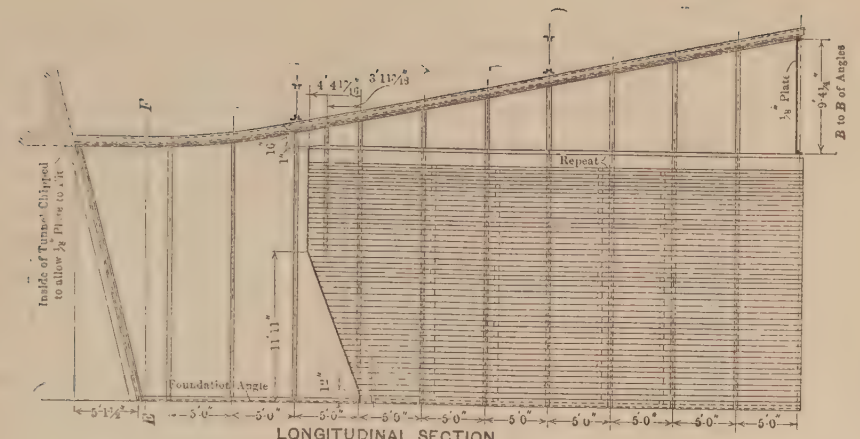
This will remove all danger from such causes, and very greatly reduce the discomfort of persons in the tunnel; but, nevertheless, in

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part C, p. 529.
† October 15th, 1904, Vol. XLIV, p. 740.
‡ January 18th, 1906, Vol LV, p. 59.

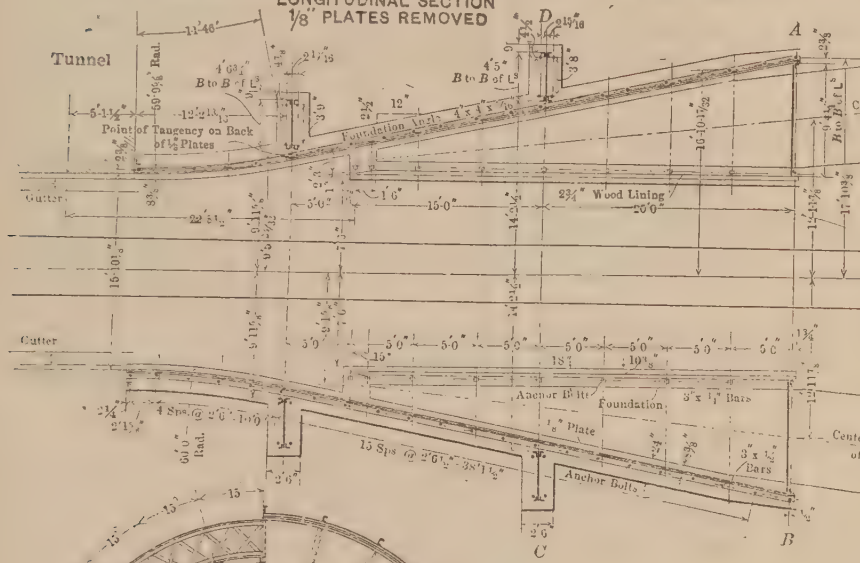
PLATE XXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVII, No. 1033.
CHURCHILL ON
TUNNEL VENTILATION.

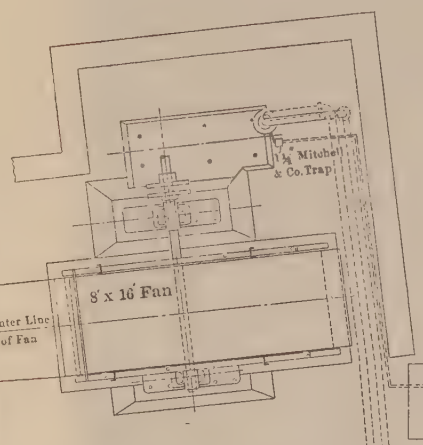
GENERAL PLAN OF VENTILATING PLANT AT GALLITZIN TUNNEL, P. R. R.

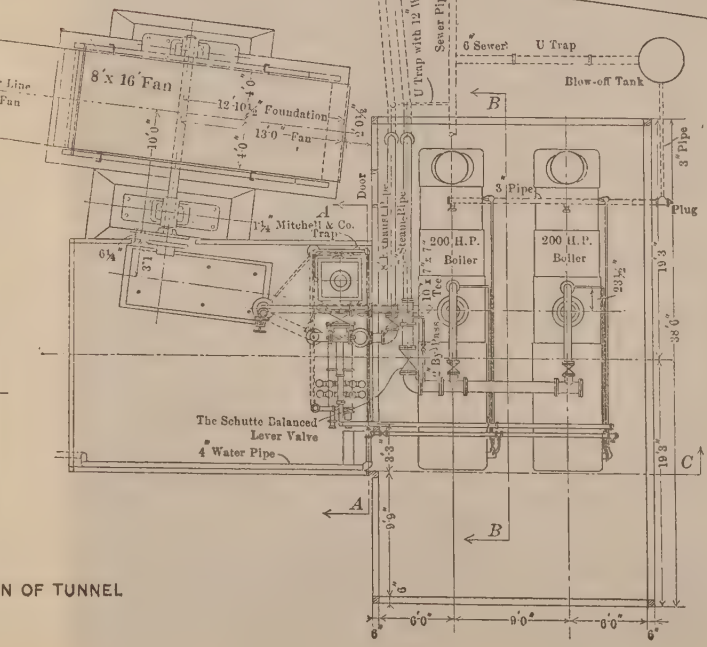
COMPLETED, APRIL 1905

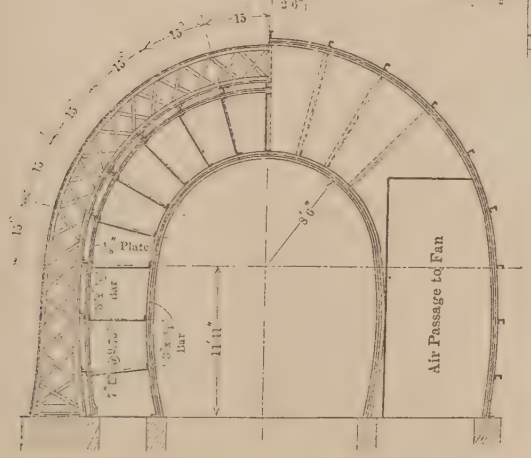


LONGITUDINAL SECTION
⅛" PLATES REMOVED



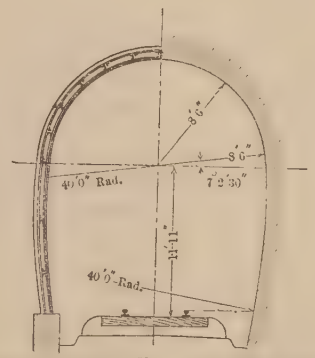






HALF SECTION C-D

HALF SECTION A-B



HALF SECTION E-F

HALF SECTION OF TUNNEL

the future some means of artificial ventilation may be found desirable, as in the case of the Mersey Tunnel, Liverpool.

Park Avenue Tunnel, New York City.—Park Avenue Tunnel, New York City, on the New York Central and Hudson River Railroad, is described in the *Transactions.** The work of providing for electrical operation, which was undertaken partly for the purpose of improving the ventilation, is rapidly progressing, under plans fully described, with official reasons therefor, in *Engineering News.*† The reasons given are, briefly: Improvement of ventilation, greater safety in operation, and increased capacity.

Simplon Tunnel.—This tunnel, which is 12.33 miles long, and is constructed for a single track, with one passing siding at the center, was completed early in 1906, and was formally opened on May 19th, 1906. The ventilating plants used during construction are described in *Transactions.*‡

When the writer visited this tunnel, in the summer of 1905, a steel sliding door at each portal was arranged so as to be raised and lowered by electric power. The plan at that time was to lower the steel door after the entrance of a train and to force fresh air from the fans at that end of the tunnel into and through the tunnel with considerable pressure.

The adoption of electricity as the power for handling trains through this tunnel has greatly simplified the problem of ventilation.

The St. Gothard Railway Tunnel.—This steam-railroad tunnel is 9.3 miles long, is located on light grades, and is double-tracked. It has a large section, the area being 603 sq. ft., or approximately 3.2 times the area of two trains occupying the tunnel section. The cars of trains while in the tunnel are lighted by gas.

This tunnel is ventilated by a Saccardo plant, at Goeschenen, at the north end of the tunnel. It consists of two large fans operated by water power. Ventilation is secured by reducing the area of the tunnel section by about 38% near its northern end.

For comparative purposes, a sample of air was taken by the writer from a car of a north-bound train, running 30 miles per hour,

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part C, p. 528.
† November 16th, 1905, Vol. LIV, p. 499.
‡ *Transactions*, Am. Soc. C. E., Vol. LIV, Part C, p. 531.

at a point about 3 miles from the south end. The temperature was 82° fahr., and the air contained 10.19 parts of carbonic acid gas (CO_2) per 10 000 volumes. The measured quantity of fresh air which the writer found to be passing into the north end of the tunnel from the ventilating plant was 413 000 cu. ft. per min. at a velocity of 686 ft. per min.

Special attention is called to the large area of the tunnel section relatively to the cars used, and the consequent small velocity and volume of fresh air used in ventilation. The other favorable condition in this tunnel is the light grade throughout its length, so that the coal consumption of locomotives while in the tunnel is kept low. At the measured velocity of the ventilating air current, a period of 1 hr. and 12 min. is required to change the air in this tunnel.

The Berlin Elevated and Electric Railway.—This electric railroad is largely above ground, but there are short sections of subway, having a rectangular section of about 231 sq. ft., in which single span there are two tracks. The stations are roomy, and each is provided with two large stairways leading directly to the surface, thus insuring fair ventilation. The air in these short subway sections was noted by the writer as being good.

Metropolitan Railway, Paris.—The Metropolitan Underground Railway of Paris is a double-tracked arched tunnel, nicely lined, and built as close to the surface of the streets as practicable. It is electrically operated. The stations are comparatively close together, averaging less than 1 700 ft. apart. The two tracks are in one span, the area of the tunnel section being 315 sq. ft. This section is greatly enlarged at each station, thus providing in one span ample room for platforms. The area of the tunnel is approximately twice that of the two trains occupying the tunnel section.

This railway is specially notable for the large tunnel section, roomy stations at short distances apart, and the provision of two large stairways at each station, giving good direct outlets for ventilation. Two short portions of two lines of this road were examined by the writer. A sample of air taken at 7.40 o'clock on a bright morning, from a Metropolitan Railway car, between Palais Royal and Alma Stations (between which there were five stops at intermediate stations), temperature 70° fahr., contained 2.94 parts of

carbonic acid gas per 10 000 volumes of air, the outside air at that time containing 1.76 parts.

The Mersey Tunnel, Liverpool.—This tunnel is 2 miles long, has a section of about 425 sq. ft., and is provided with two tracks. It is now electrically operated, and is well ventilated by large fans.* The air in this tunnel is now good. A sample taken by the writer from a car in this tunnel, July 24th, 1905, contained 4.70 parts of carbonic acid gas per 10 000 volumes.

The District Railway, London.—The District Railway is a double-tracked subway, with its grade level generally as close as practicable to the streets above it, and with frequent openings to the surface, in addition to having stations approximately ½ mile apart. It is operated by steam, but it is being gradually electrified, primarily to secure better ventilation.

For comparative purposes, the writer quotes a test, made in 1902 for the London County Council, which showed that the air in a car contained 15.1 parts of carbonic acid gas per 10 000 volumes. A sample of air taken from a District Railway car by the writer, between Mansion House and Temple Stations, on a fair day, June 27th, 1905, at 4.25 P. M., temperature 76°, contained 11.27 parts of carbonic acid gas per 10 000 volumes, the outside air at that time containing 3.55 parts.

Metropolitan Railway, London.—The Metropolitan Railway is similar to the District Railway in construction and operation, and is also being gradually electrified, for the same reasons.

A test made in 1902 for the London County Council showed that the air in a car on this road contained 28.8 parts of carbonic acid gas per 10 000 volumes. A sample of air taken from a Metropolitan Railway car by the writer, between Gower Street and Portland Road Stations, on a very clear day, July 24th, 1905, at 12.40 P. M., temperature 74°, contained 14.07 parts of carbonic acid gas per 10 000 volumes, the outside air at that time containing only 1.75 parts.

City and South London Railway.—This electrically operated line extends from Angel to Clapham Road, about 6 miles, with thirteen stations on the entire line, the average distance apart being ½ mile.

* A detailed description may be found in the papers on the Ventilation of Tunnels, International Engineering Congress, St. Louis, 1904, *Transactions*, Am. Soc. C. E., Vol. LIV, Part C, pp. 527–558.

It consists of two single-track tubes, the inside diameter of each being 10 ft. 6 in. They are deep below the surface of the ground. As these tubes are small, the clearance around the trains is very narrow. The tubes are connected by small passages at the stations and by cross-overs near a few of them. The area of each tube is 86 sq. ft., the area of both tubes being, therefore, 172 sq. ft.; and one tunnel section equals about 1.4 times that of a train. The travel on this line is not as heavy as on some others, notably the Central London Railway. It is not ventilated artificially, and the air is not as good as in other tubes of the electric railroads in London. The only natural ventilation is through a stairway and generally three lifts at each station. The stairway has a minimum opening of from 35 to 40 sq. ft. One lift contains about 2 500 cu. ft., and has a capacity of about fifty persons. During ordinary light traffic only one of these lifts is required per train.

The draft produced in the tunnel by an approaching train lasts only 1 min., and amounts to only about 17 000 cu. ft. The draft that follows a train lasts about ½ min., and amounts to only about 8 000 cu. ft. At the stairway of Borough Station, at the time of the passage of a train, a down-draft was noted for 1 min., equal to about 7 000 cu. ft. of air, followed by an up-draft for ½ min., amounting to about 2 000 cu. ft. of air. In addition to this, each lift removes in its upward passage about 2 500 cu. ft. of air. This quantity per station is clearly insufficient to balance the vitiation of the air by the occupants of a train.

Tests made in 1902 for the London County Council showed that there were 14.2 parts of carbonic acid gas per 10 000 volumes of air in an empty carriage. A sample of air taken from a City and South London car by the writer, between London Bridge and Moorgate Stations, on a fair day, June 27th, 1905, at 3.45 P. M., temperature 74° fahr., contained 11.78 parts of carbonic acid gas per 10 000 volumes, the outside air at that time containing 3.55 parts.

Another sample, taken on July 24th, 1905, from a car of the same road between Elephant and Castle Station and London Bridge Station on a still brighter day, at 9.25 A. M., temperature 69° fahr., contained 8.69 parts of carbonic acid gas per 10 000 volumes, the outside air at that time containing only 1.75 parts.

The sample taken in June shows that the car air contained a

little more than three times the quantity of carbonic acid gas found in the outside air on that day, while the sample taken on July 24th, though lower in this gas, contained nearly five times the quantity found in the fine outside air on that day, and actually seemed to the sense fully as poor in quality as that taken in June.

Central London Railway.—This electrically operated line extends from Shepherds Bush to the Bank, about 6 miles, with thirteen stations in this distance, averaging ½ mile apart. It consists of two single-track tubes, each 11 ft. 6 in., minimum inside diameter, connected by passages at the stations, also by six cross-overs in the entire length. The area of each tube is 100 sq. ft., the area of both tubes being 200 sq. ft. The cubical contents of the entire railroad tubes are equal to about 6⅓ million cu. ft.

The natural ventilation is by a narrow stairway and about three lifts at each station. Mechanical ventilation of the tunnels is secured by a fan at Shepherds Bush Station, which draws in fresh air from the opposite end of the line—that is, Bank Station, and other stations at intermediate points. The fan is 5 ft. wide, 20 ft. in diameter, and makes 145 rev. per min. It is operated by electricity, and is rated at 300 h. p. Its capacity is 100 000 cu. ft. per min. It is operated from 1.00 to 4.00 A. M. (3 hr.), with the outlet doors at all intermediate stations shut, therefore it draws in fresh air from Bank Station. During the operation of the fan, the water-gauge pressure in the tunnel is 4½ in., and the linear velocity of the air in the tunnel is 500 ft. per min. The fan is also operated during each day from 11.00 A. M. to 5.30 P. M., or 4½ hr., all the doors of the passageways at the stations being open. This improves the air in the tunnels for about one-half the length of the line—that is, for the 3 miles nearest to Shepherds Bush Station—it also clears out the cars as they come to this last station.

The result of night pumping, as indicated by an anemometer placed in the tunnels at the Post Office Station (the first one west of Bank Station), shows that each week an average of 123.9 million cu. ft. of fresh air entered during the night pumpings, which is equivalent to 17.7 million cu. ft. per night. As the tunnels contain 6⅓ million cu. ft., they are cleared about 2¾ times each night.

The result of this night ventilation is shown by tests taken on February 15th, 1905, at the Marble Arch Station cross-over siding,

which is about half way between Shepherds Bush and Bank Stations. At 9.30 P. M. the air was found to contain 10.3 parts of carbonic acid gas per 10 000 volumes. The fan was started at 1.37 A. M., and at 2.15 A. M. the air at the same point contained only 3.7 parts, which was the same as that in the outside air.

The amount of natural ventilation procurable is shown by some other tests made during the same night. Between 9.30 P. M. and 1.00 A. M. (3½ hours), the air in the tunnel had cleared slowly from 10.3 parts to 6.0 parts of carbonic acid gas per 10 000 volumes at the latter time. During these tests the temperature of the tunnels was 66½° fahr. Another test, taken on February 2d, 1905, with the fans not running, and when the outside air contained 4.4 parts of carbonic acid gas per 10 000 volumes, showed that the tunnel air averaged 7.07 parts of carbonic acid gas, while the air in a full car contained 10.7 parts.

The methods now adopted for ventilating the Central London Railway have been put into force since 1902, and the question of adding to this ventilating system is now being considered.

The first ventilating plant on this road was at Bond Street Station, but it has not been used since the installation of the present ventilating plant at Shepherds Bush.

A long series of tests made in 1902 for the London County Council showed that the air in the cars contained a minimum of 9.60 parts of carbonic acid gas per 10 000 volumes, at a time when the outside air contained 3.30 parts, and a maximum of 14.7 parts of carbonic acid gas per 10 000 volumes, when the outside air contained 3.30 parts.

A sample of the air taken from a Central London Railway car by the writer, between Chancery Lane and Post Office Stations, on a fair day, June 27th, 1905, at 1.55 P. M., temperature 76° fahr., contained 5.92 parts of carbonic acid gas per 10 000 volumes, at a time when the outside air contained 3.55 parts.

Another sample taken by the writer on July 24th, 1905, from a car on the same road, between Tottenham Court Road and Oxford Circus Stations, on a still brighter day, at 1.00 P. M., temperature 72½° fahr., contained 7.04 parts of carbonic acid gas per 10 000 volumes, the outside air at that time containing only 1.75 parts.

Great Northern and City Railway, London.—The Great North-

ern and City Railway extends from Moorgate Station to Finsbury Park, a distance of 3.43 miles. It is electrically operated and lighted, and there are six stations, varying from 2 000 to 6 200 ft. apart. Drayton Park Station, which is 4 200 ft. south of the northern terminus, is all in the open for 600 ft., therefore the main tunnel on this road extends from Drayton Park to Moorgate, a distance of 2.62 miles; the longest distance between stations being from Essex Road to Old Street, 6 200 ft. This line was put into operation in 1904, and is in many respects the best one examined by the writer. It consists of two single-track tubes, each 16 ft. inside diameter, connected by five small cross-passages at each of the station platforms, in addition to some cross-overs. The station platforms are generally 420 ft. long, to accommodate a standard train of seven coaches each 50 ft. long, carrying a total of 430 passengers. The special features of this subway are its large tubes and the large stairways and passageways, as compared with other London subways. Each of these items is claimed by the officials of the road to be a direct benefit in ventilation. The line is expected to handle 27 000 000 passengers per annum. The maximum train service arranged for is a 3-min. spacing, making 3½ miles in 13½ min., including 20 sec. for each of four intermediate stops.

The area of each tube is 190 sq. ft., the area of the two tubes being 380 sq. ft. The area of the tunnel, therefore, is about 1.8 times that of the large cars of the trains therein. The approximate cubical contents, from the open end at Drayton Park to the closed end at Moorgate Station, is 5½ million cu. ft.

Its natural ventilation is by two stairway passages at each station, the minimum area of each passage being 57 sq. ft., or a total of 114 sq. ft., in addition to that furnished by three lifts at every station, each having a floor area of 130 sq. ft., or cubical contents of about 1 500 cu. ft.

Its artificial ventilation, applied since the line was opened, is obtained by a fan, having a diameter of about 6 ft., located at Pool Street, which is 6 330 ft. from the Moorgate end of the tubes. This fan has a capacity of 1 000 000 cu. ft. of air per hour, and, as operated, forces fresh air through a heading 10 ft. in diameter into the up-tunnel—that is, toward Moorgate Station, in the direction of travel. A large part of this current travels to Moorgate Station

cross-over and then into the down-tube. The present practice is to operate this fan twice each day, during the busiest periods, from about 8.00 to 10.00 A. M., and from 5.00 to 8.00 P. M., so that a volume of fresh air practically equal to the cubical contents of the entire subway is forced into it each 24 hours.

As to natural ventilation: Measurements of the currents in the stairways on the arrival and departure of trains show approximately a total of down-currents for each train per station of 20 000 cu. ft., and about the same for up-currents. The total ventilation per train for each direction, therefore, amounts to about 90 000 cu. ft. On a 5-min. spacing of trains, each filled with 430 persons, the change of air per hour amounts to 1 080 000 cu. ft. in the entire subway, or 18 000 cu. ft. per min. However, as there are constantly four trains in the tubes carrying a maximum of 1 720 people, and as each person requires 30 cu. ft. of air per min., the total requirement is more than 51 000 cu. ft. of fresh air per min., in order to keep the tunnel air of the same quality as the outside air, or about 26 000 cu. ft. per min. in order not to increase the carbonic acid gas by more than twice the quantity in the outside air. These figures show why ventilation has been found necessary during the busy hours to maintain air of the desired purity even in this excellently constructed subway.

On March 5th, 1904, before ventilation was established, a test of the air in this subway was made by Dr. Critchley, with the following results: The air in the street was found to contain 4.5 parts of carbonic acid gas per 10 000 volumes, that of a car in the subway was found to contain 9.0 parts, the temperature being 61½° fahr.

On February 20th, 1905, Dr. Southerland found that the air on the platform in the subway at Old Street Station contained 4.48 parts of carbonic acid gas per 10 000 volumes, temperature 54° fahr., while that in a car approaching this station was found to contain 7.22 parts, the temperature being 55½° fahr.

A sample of air taken from a Great Northern and City Railway car, by the writer, between Highbury and Old Street Stations, on a very bright day, July 24th, 1905, a little after noon, temperature 68° fahr., contained 2.35 parts of carbonic acid gas per 10 000 volumes, the outside air at that time containing 1.75 parts.

The Rapid Transit Railway, New York City.—The subway sections of this railway are either four-tracked, of rectangular cross-section, or of two double-tracked cross-sections. The former is the one used on the busiest part of the line, below Forty-second Street. The stations on this portion average about 1 500 ft. apart. The area of this four-tracked cross-section is about 650 sq. ft. The sectional area of the tunnel therefore is 1.8 times that of the four trains occupying the subway.

This subway is of steel construction, with concrete, and is built quite close to the surface of the streets. The outlets to the street at the stations consist of from two to a maximum of eight stairways, which, while large enough for the people entering the stations, are not as direct as in some of the European subways, and have not a total outlet area as large in proportion to the tunnel cross-section, or to the number of trains and people in the subway at one time.

The writer predicted in 1904 that mechanical ventilation would be found necessary to secure a positive change of air.

On November 21st, 1904, a sample of air was taken from a car in the Rapid Transit Subway, between Forty-second and Twenty-eighth Streets, and one between Spring Street and Astor Place, and these were found to contain too great a percentage of carbonic acid gas. Again, on December 28th, 1905, at 4.00 P. M., the temperature being 60° fahr., a sample of air, taken from a car of the Rapid Transit Railway, between Twenty-third Street and Brooklyn Bridge Stations, was found to contain 15.58 parts of carbonic acid gas per 10 000 volumes of air.

The various tests of air made by the writer in London and Paris have shown that, except in the deeper subways, the air has as high a temperature as that outside. In the Rapid Transit Subway, during the summer of 1905, a considerably higher temperature of the air was recorded. This is another reason why it will become very urgent to establish a complete system of mechanical ventilation, for the health of the employees and the comfort of the patrons, as in the case of the subway tunnels in Boston, Liverpool, and London.

Subway Tunnels Generally.—Among the statements heretofore made are included records of the determination of carbonic acid gas (CO_2) in the London subways as well as in their cars. From

these data, also from similar tests of air reported upon by the London County Council, in subway tunnels when they were not fully ventilated, the air found in subway cars contains about 45% more carbonic acid gas than the average air found in the subway tunnels and stations. Inasmuch as the patrons and a large number of the employees of a subway tunnel line spend most of their time in the cars, the vital fact to be determined, as to the health and comfort of these individuals, is the amount of carbonic acid gas in the cars themselves. Therefore, tests of air in subway tunnels and stations are simply somewhat indicative of the conditions that will be found in the cars. It may be anticipated that the amount of carbonic acid gas in the cars will be at least 45% more than in the subway tunnel, and in crowded cars the proportion of carbonic acid gas will be still greater.

The writer, therefore, has confined his investigations everywhere to the vital question, namely, the quantity of carbonic acid gas (CO_2) found in the ordinary cars of tunnels, and the tests by him were taken in cars ranging from one-fourth to one-half full. Any reports of the amount of carbonic acid gas found in subway tunnels or stations, therefore, should be increased by about 45% in order to obtain any fair idea of the conditions met by the passengers and employees themselves; so that, if a subway contains as much as 6.5 parts of carbonic acid gas per 10 000 volumes of air, there will probably be found in the cars 8.4 parts or more.

It has further been shown by detailed measurements of natural ventilating currents in the London subway tunnels, from the smallest to those of the largest section, that an insufficient quantity of air is drawn into them to ventilate all parts of them properly. While the portions having light traffic may at times be free from too large a proportion of carbonic acid gas, those parts having heavy traffic will contain an excess. The conditions of traffic in the Rapid Transit Railway of New York, for example, are such as to indicate the presence of much more carbonic acid gas in the cars while traversing the lower portions of the subway tunnel than in the remainder.

Proper ventilation is not secured unless all parts of a subway tunnel receive a sufficient quantity of fresh air to balance properly the vitiation of that in the cars by the occupants thereof, and this

result cannot be secured, as has been shown, except by an efficient system of mechanical ventilation that will distribute fresh cool air everywhere in the cars, as well as in the subway tunnel and its stations.

Standard of Purity For Air.—Much has been written as to what constitutes good air and what is dangerous. The most important test is the determination of the parts of carbonic acid gas (CO_2) in a given sample per 10 000 volumes. Ordinary air varies considerably, during different kinds of weather, from about 1½ parts to more than 4 parts of carbonic acid gas per 10 000 volumes, but the ordinary figure is about 4 parts. The best authorities state that the air in any subway should not contain more than twice the amount of carbonic acid gas found in the outer air (the two being nearly always directly compared), and that, therefore, the general limit is 8 parts of carbonic acid gas per 10 000 volumes. The "Factory Act" of England makes 9 parts of carbonic acid gas as the maximum in rooms, etc. The report to the London County Council in 1902 places the safe limit at 8 parts, as being the maximum that should exist in the air of subways; and that report states further that the number of bacteria in subway air was generally found to increase and decrease in proportion to the carbonic acid gas in the air. The writer, while investigating this subject in London, during July, 1905, learned that public opinion is very strong in demanding air in subways that shall generally not contain more than 8 parts of carbonic acid gas per 10 000 volumes of air, and that periodical attacks by the press are made, substantiated by detailed reports of noted chemists, against any subway tunnel where, for a period, air is found to contain more than this quantity of carbonic acid gas. Hence, it may be stated that 8 parts of carbonic acid gas per 10 000 volumes is the maximum proportion of carbonic acid gas that should be allowed in the air of cars in subway tunnels.

DISCUSSION.

Mr. Rice. GEORGE S. RICE, M. AM. SOC. C. E. (by letter).—Mr. Churchill has referred to the conditions in the New York subway. At a later time the writer hopes to present to the Society a comprehensive paper on this subject. At present, however, it seems proper to mention a few facts concerning the "so-called ventilation" of the subway. The words "so-called ventilation" are used advisedly, because there seems to be a very general misconception of the subway problem.

Heretofore, the objectionable conditions which have prevailed in underground railroads have been due mainly to the vitiation of the air by locomotives. The conditions were improved by providing mechanical means for removing the bad air and replacing it with fresh air from the outside. When electrical motors were substituted, all except a few thousandths of 1% of the foul air was at once removed. It only remained to supply enough fresh air for the consumption of the passengers. To accomplish this, the train movement was depended upon, in seven of the nine subways which have been constructed in America and abroad.

These roads were so near the street surface that it was thought that ample renewal could be obtained from the piston action of the trains. Where the roads were short, or the cross-sections large, and the openings into the outside air numerous, the results have been good, notably in Berlin and Paris. In most other cases, experience has indicated that wholly satisfactory conditions cannot be obtained in this manner. The tube roads are usually constructed as two single-track tubes, the trains thus traveling in one direction in each tube. Even under such favorable circumstances, the piston action of the trains has not produced a satisfactory air renewal, and mechanical ventilation has had to be resorted to.

The New York subway (the deep tunnel portions of the road are not referred to in this discussion) is near the street surface, its cross-section is large, and the openings are numerous. These conditions favor ample air renewal from train movement.

From careful investigations which have been made, it has been found that the subway air, when measured by the carbon dioxide standard, is all that could be desired. The average of more than 2 000 analyses of air samples taken in the streets, in the stations, in the subway between stations, and in the trains, shows: in the subway only 4.81 volumes of carbon dioxide in 10 000 volumes of air; in the streets, 3.67; difference, 1.14 volumes. The greatest amount found in the subway was 8.89 volumes in 10 000.

In winter it is to be expected that the air in the trains will sometimes be high in carbon dioxide. The car doors and windows are

then closed, and often the guards cannot keep the ventilators open on account of objections from passengers. Under such circumstances, the same vitiation would exist if the trains were running in the open air. Therefore, such conditions cannot be charged to poor ventilation of the subway. But, on account of several peculiar characteristics, as a result of construction and operation, certain conditions have developed in this subway which do not exist to the same degree in any other subway or tube railroad now in operation. These characteristics and the resulting conditions are as follows: Mr. Rice.

1.—The Large Amount of Heat Liberated in the Subway from all Sources, due to the Operation of Trains.—During the summer of 1905, this heat approximated an average of 25 285 000 B. t. u. per hour, throughout the 24 hours, varying from 4 689 000 to 42 400 000 B. t. u. per hour, approximately.

The greater part of this heat is dissipated through the subway structure into the adjoining ground. A relatively small amount is carried out of the openings of the subway by the exhaling and inhaling action of the trains. About 93% is dissipated in these two ways. The remaining heat increases the subway temperature during summer to an average of about 6° greater than that of the outside air; or, approximately, to an average of 15° greater than the temperature in the subway before operation.

2.—The Effect of the Subway Structure, in Location and Construction, upon the Heat Dissipation.—The amount of heat dissipated through the subway structure is dependent upon the difference in temperatures between the subway and the ground around it—or heat head—the ground-water level, relative to the structure, and the conductivity of the structure. In summer, the temperature of the adjoining ground decreases with the increase of the depth of the subway below the surface. Ground temperatures at some distance from the subway, observed on August 31st, 1905, at noon, were as follows:

At	2½	ft.	below	the	surface,	75° fahr.
"	6	"	"	"	74°	"
"	10	"	"	"	72°	"
"	15	"	"	"	68°	"
"	20	"	"	"	66°	"
"	30	"	"	"	60°	"

At the same time the temperature in the subway at the nearest point was 86°, and the temperature of the outside air was 82 degrees.

Other things being equal, it appears that, with the same amount of heat being liberated in the subway, the heat head is greater the greater the depth of the subway below the surface. In consequence

Mr. Rice. of this, more heat is dissipated and the interior temperature is lowered.

Ground-water, on account of its high specific heat and continual movement, maintains a relatively greater difference between the inside and outside temperatures, and thereby produces a cooler condition in the subway.

The conductivity of the structure is largely affected by the water-proofing. It has been estimated that it reduces the conduction through the New York subway structure at least 33 per cent. Of the seventeen most important subway and tube railroads, only two are completely surrounded by a water-proofing envelope; eleven have no water-proofing at all, the other four are partly water-proofed. The New York and Berlin subways are wholly water-proofed. The subway portion of the former is 10½ miles long; of this, 6½ miles are four-tracked. At any time during the maximum operation, 231 cars are in the four-track portion of the subway, carrying approximately 23 000 passengers.

The Berlin subway is a two-track road; it is a little more than one mile long, and during maximum operation it contains only 12 cars at any one time, carrying 2 100 passengers.

Therefore, with regard to the heat condition, both in the amount generated and the loss through the structure, it can be said that no other underground railroad in the United States, in Great Britain, or on the Continent, is comparable with the New York subway.

3.—The Iron Dust Liberated by the Wear of the Rolling Stock and Rails.—This dust comes mainly from the brake shoes. Between the Brooklyn Bridge and Ninety-sixth Street Stations the brakes are applied about one-quarter of the total time consumed in the run of the local trains, and one-eighth of the total time of the express-train runs. More than 20 tons of these shoes are actually ground to dust in a month on the whole road.

From what has been said, it must readily appear that the subway problem is not simply one of ventilation, as ventilation is usually understood. If the air between the Brooklyn Bridge and Ninety-sixth Street Stations were renewed every 3 hours throughout the 24 hours, during the maximum day's traffic which has yet occurred, the carbon dioxide should not exceed 8 volumes on the average; or, half-hour renewals during the rush hours would maintain the same standard during that time. Such renewals would remove a material quantity of the iron dust, but would have a relatively small effect in reducing the temperature. No amount of air renewal could make the subway cooler than the outside air. In fact, with 3-minute renewals, the subway would probably be a degree or more warmer than the outside air at all times in summer.

To improve the conditions in the subway, construction is now under way to renew the air in the four-track portion about every

25 minutes, in addition to such renewal as now takes place, due to the exhaling and inhaling action of the trains. This is to be accomplished during the day by the train movement actuating movable louvers, or valves, and at night by motor-driven fans. Mr. Rice.

The louvers and fans are to be installed in ventilating chambers constructed at points midway between the stations. The air is to be exhausted from the subway at these points. The fresh air will come in at the stations, where about 5 000 sq. ft. of vault lights have been cut out and replaced with gratings to provide a large additional inlet opening.

Bringing about the renewal in this manner has several advantages. It is accomplished during the greater portion of the day by the train movement acting through the louvers, thereby reducing the cost of operation. At night, when the train movement is small, and the outside temperature lowest, the fans remove the air, thereby cooling off the structure to some extent.

In case of accident stalling trains, or fire in the subway, the fans are available to provide fresh air, or to remove the smoke; the smoke being drawn away from the station points and thus preventing such an accident as occurred in Paris. The chambers also provide entrances and exits to the subway at points midway between stations.

When the work described is completed there will be a material improvement in the subway, but it will not be cooler than the outside air. If the public insists upon a cool subway, cold air will have to be introduced. The method of air renewal outlined will enable this to be done in the most economical and satisfactory manner.

R. P. Bolton, M. Am. Soc. C. E.—This paper contains some very interesting lessons for all engineers, especially for those who are dealing with the ventilation of the New York subway. Special interest attaches to the proposed remedy for the defects in the present condition of affairs, which is generally felt to be almost intolerable. The apparatus now being installed is expected to improve the atmospheric condition of the subway by two means: First, the train action is to be depended upon to force the dense air out through louvers in side chambers, and at night fans are to force fresh air into the interior and displace the dense air. Train movement, however, has already demonstrated its inability to act as a motive agent, to any great extent, and it is hopeless to expect results from that source of air movement. The fans installed in the side chambers will, no doubt, effect beneficial results, if their proportions are sufficiently ample, and if they are run at all times. Mr. Bolton.

It would seem that the lessons learned by experience in subways in other countries, the results of which are presented in this paper, could have been applied to the New York subway ere this. This

Mr. Bolton. whole subject was presented to the engineering world through this Society by Mr. Churchill, in 1904, in a very comprehensive manner, and, from the facts he had collated, he then urged the view that the New York subway would require the installation of special means of artificial ventilation, and so it appears that the end of the year 1906 is a very late date for the authorities to be engaged on this matter, and to be as yet undecided as to the methods and results. In the public interest, it might be hoped that the knowledge and energy of the author, who has so closely studied the subject, and has favored the Society with this paper, might be engaged upon the work, and be devoted to giving the public prompt relief.

With regard to the effect of the closely fitting trains in the underground tubes in London, the speaker's observations two years ago were that incoming trains pushed a considerable draft into the stations, the air moving up through the elevator shaft and streetways, but that the same air was drawn back when the trains reentered the tunnel; and he felt then, as he feels now, that train movement is by no means to be depended upon for effective air renewal in any kind of subway.

The subject of effective air replacement and the dilution of the dense and heated air in the tunnel is of very wide nature, and it is not possible, without very full details and statistics of the various parts of the tunnel, to point out the methods and capacities required to effect the desired improvement. It would be of much interest to learn something of the capacities of the fans now being installed in the New York subway and their relation to the contents of the part of the tunnel they are to affect.

While the subject of tunnel air dilution is shown by the paper to be a matter in which wide variations may exist, one feature appears to be common to all such lines of transit, namely, the proportionally defective condition of the atmosphere inside the cars.

This feature is nowhere more pronounced than in the crowded cars of the New York subway, and it has seemed to the speaker strange that no effort has been made to effect any improvement in this matter, as it would afford a very direct relief to the traveling public.

In point of fact, the discomforts of the public are not so much due to the condition of the air in the tunnel as to the aggravation of that condition inside the cars. The opening of the forward doors of the head cars of trains in the summer, which measure was ordered last year, has been greatly appreciated by many suffering travelers. The air may be heated and dense, and is undoubtedly deficient in oxygen and overcharged with $C\ O_2$, but its rapid movement through the car is pleasant and cooling. The effect, however, is lost in the following car, on account of the restricted area of the

doors and vestibules. The open windows are not effective, as the air of the tunnel rolls over the surface of the sides of the car. Mr. Bolton.

It is noticeable that these cars are provided with head vents of the same character as those on the cars of surface railroads, and these are as ineffective as elsewhere, and are equally unsuitable to the subway conditions.

There appears to be no practical reason why the cars should not be provided with ample air scoops, so that every car should be afforded a thorough air passage through it, which can be modified for winter conditions in such a way that the dilution of the interior atmosphere with C O_2, at least, may be kept, approximately, the same as the body of the tunnel.

The speaker's observations show that these interiors are often far worse in winter than even under summer conditions, on account of the closing of doors, windows, and vents, ineffective as the latter are.

The application of agitators to the car interiors might be a source of relief for winter conditions, but should be accompanied by a positive removal of the dense air from the lower part of the car interior, and its replacement by the lighter air from overhead.

One of the contributing sources of discomfort is the heating up of the car itself by the emitted heat of the motive apparatus and brakes, the heat of the interior lighting, as well as the heat emitted by the passengers themselves.

The course suggested, therefore, would relieve the difficulty and discomfort at the source, and has the advantage, over the extensive arrangements required to affect a change of the air in the tunnel itself, of a local simplicity and directness.

CHARLES S. CHURCHILL, M. AM. SOC. C. E. (by letter).—It is gratifying to have recorded, by both Messrs. Rice and Bolton, a concurrence in the writer's statements to the effect that the piston action of the trains, even under favorable circumstances, does not produce a satisfactory air renewal in electrically operated subways, and that, therefore, mechanical ventilation is necessary. Mr. Churchill.

Soon after the completion of this paper, the writer received a copy of the report of George A. Soper, M. Am. Soc. C. E., on "The Condition of the Air of the New York Rapid Transit Subway." The diagrams showing parts of carbon dioxide per 10 000 volumes of air in the subway close after a very rapid rising scale on December 25th, 1905, with nearly 7 parts of carbon dioxide; but the statement is added that as high as 8.89 parts of carbon dioxide were found in the subway near the Grand Central Station on December 27th, 1905, which determination is also referred to by Mr. Rice. It does not seem strange, therefore, that on the following day, December 28th, 1905, in the section of the subway further down town,

Mr. Churchill. where general conditions are not so good, the writer found in a car, not more than half full, with ventilators open, 15.58 parts of carbon dioxide per 10 000 volumes, which is certainly far from satisfactory.

The parts of carbon dioxide per 10 000 volumes of air is indicative of the number of bacteria and all classes of impurities existing in a subway. In other words, while the percentage of carbon dioxide may appeal to a chemist, there are many other items which appeal to the engineer and traveler to just as great an extent.

Some London experts called the writer's attention to the peculiar and disagreeable odor found by him in the unventilated tunnel of the City and South London Railway, which was also found to be relatively high in contents of carbon dioxide; and in September, 1906, while traveling on an express train through the New York Rapid Transit Subway, between Grand Central and Fulton Street Stations, in a car with both ventilators and windows open, the writer and others noted those odors indicative of insufficient ventilation.

The mechanical ventilating system of the Boston Subway is proportioned so as to secure a renewal of air in all parts of the subway during intervals of about every 10 min.

While the residents of New York, and the traveling public generally, should be, and undoubtedly are, grateful, that plans for the mechanical ventilation of the New York Rapid Transit Subway are actually being carried out, after an agitation of the subject for two years, it is to be hoped that the work may be thoroughly done; but the plan of Mr. Rice for a mechanical renewal of air in 25 min., as compared with 10-min. intervals provided for in the Boston Subway, would indicate the proposed use of fans of smaller capacity than may subsequently be found advisable.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
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TRANSACTIONS.

Paper No. 1034.

CONCERNING THE INVESTIGATION OF OVERLOADED BRIDGES.*

By Wilbur J. Watson, M. Am. Soc. C. E.

With Discussion by Messrs. R. D. Coombs, Mansfield Merriman, Henry B. Seaman, George B. Francis, Theodore Belzner, J. E. Greiner and Wilbur J. Watson.

For some years a large part of the writer's professional work has been the investigation of existing bridges which are subjected to loads greater than those for which they were originally designed; and, in the course of this work, he has found, among bridge engineers, such a divergence of views regarding the maximum unit stresses, etc., which may be safely allowed in an existing structure, that he takes this opportunity of bringing the matter before the Society for discussion, in the hope that thereby greater uniformity of opinion may be established. The rapid development of electric traction lines, both urban and interurban, many of which are built upon highways, has had the effect of overloading many highway bridges.

The writer has in mind an interurban line which was built upon a highway and crossed many bridges. The engineer in charge, who was not a bridge engineer, reviewed the stresses caused by the new loading, and condemned every structure in which they exceeded those recommended by a certain standard specification for highway

* Presented at the meeting of September 5th, 1906.

bridges. On the other hand, lines have been built over old highway bridges without any investigation whatever being made as to the strength of the bridges. It is hardly necessary to state that such lines were built by promoters, without the aid of competent engineers.

One promoter, desiring to show his contempt for engineers in general, told the writer that he had built an interurban line, ten miles in length, and that his total expense for engineering services amounted to $75. The writer having ridden over this line, sees no reason to doubt the promoter's statement.

It is the writer's practice, when called upon to report concerning the safety of an existing structure, to make a careful examination of the structure in the field, in order to determine the following points:

First.—The general dimensions;

Second.—The sizes of the members, connections, etc., making full allowance for corrosion, if any;

Third.—The character of the material, whether steel or iron;

Fourth.—The workmanship;

Fifth.—The presence of, or liability to, secondary stresses.

The stresses are then computed, and to the dead-load and live-load stresses an amount is added to cover those due to impact, vibration and similar secondary stresses. In computing this allowance for secondary stresses, the rational impact formula, $I = L \frac{L}{L + D}$, is used, in which I = the allowance to be made for secondary stresses; L = the stress due to the live load considered as a static load; and D = the stress due to the dead load only. In case there exist secondary stresses due to eccentric connections, which are very common in old highway bridges, or due to other defects in design or execution, these should be added to obtain the maximum.

In regard to allowable unit stresses, it may be stated, as a general principle, that the greatest possible stress in any member should not exceed the elastic limit of the material.

There are two reasons why this limit has been used:

First.—In exceeding the elastic limit, the stress causes a permanent distortion of the member, which, in turn, causes a change in the conditions which were assumed in computing the stresses.

Second.—The classical experiments of Wöhler and Bauschinger demonstrated that repeated strains greater than the elastic limit of the material, but much less than the ultimate strength, might cause rupture.*

It is well known that eye-bars and built-up members do not develop the full strength indicated by their sections and by tests on small-sized pieces, and it would appear that it is not safe to assume the elastic limit of full-sized members to be more than about 75% of the elastic limit indicated by tests on small test pieces of the material of which these members are composed. It has also been demonstrated that single angles, connected by one leg only, cannot be depended upon to develop more than about 60% of their computed strength.†

The second cardinal principle, then, as used by the writer, may be stated as follows: The greatest computed unit stress in any member in tension, making full allowance for ordinary and extraordinary secondary stresses, should never exceed 75% of the elastic limit of the material for members symmetrically connected, nor 60% of the elastic limit of the material for angles connected by one leg only. The latter case is very often found in old lattice trusses. This would give, for symmetrically connected members in tension, working stresses of, approximately, 20 000 lb. per sq. in. for iron, 24 000 lb. per sq. in. for soft steel, and 26 000 lb. per sq. in. for medium steel.

In examining old bridges, it is necessary to note carefully the construction of the compression members, as they are often insufficiently stiffened, especially the T-sections formerly used to such an extent. The writer will risk the statement, drawn entirely from personal knowledge, that more failures occur in compression members than in tension members, and he has in mind the failure of an electric railway bridge by the buckling of the top chord (a T-section) under a unit stress in the steel not exceeding 22 000 lb. per sq. in. Full allowance for impact, etc., as outlined above, had been made, and the failure was entirely due to the lack of proper stiffening of the member.

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* Paper by H. B. Seaman, M. Am. Soc. C. E., on "The Launhardt Formula, and Railroad Bridge Specifications," and discussion thereon by C. C. Schneider, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XLI, pp. 140 and 173.

† Paper by J. E. Greiner, M. Am. Soc. C. E., on "Tests of Bridge Members," *Transactions*, Am. Soc. C. E., Vol. XXXVIII, p. 41.

It is the writer's practice to increase the stress in compression members by the following formula, $S_2 = S_1\left(1 + \frac{l^2}{18\,000\,r^2}\right)$, in which S_1 = the direct unit stress, S_2 = the maximum unit stress, l = the length of the member, and r = the radius of gyration of the member, in inches. This will be recognized as Gordon's formula as ordinarily used for pin-connected columns.

For computing the stresses in an improperly designed column, no general method is offered, as it is governed by the peculiarities of each case. It is in cases such as these that good engineering or structural judgment is most needed.

Many structures are unnecessarily condemned on account of rivets overstressed in shearing and bearing. The usual assumptions governing such cases are as follows:

First.—That the shearing strength of the material is equal to two-thirds of the tensile strength, and that the bearing strength is equal to twice the shearing strength;

Second.—That the entire stress transferred from one member to another must be considered as transferred entirely by the rivets in shearing and bearing, making no allowance whatever for the friction between the surfaces of the members;

Third.—That the diameter of the rivet is assumed as the diameter before driving.

These assumptions would give maximum allowable stresses for iron rivets of, approximately, 13 300 lb. per sq. in. in shear, and 26 000 lb. per sq. in. in bearing. The writer's experience has convinced him that riveted connections will not fail under any such limits of stress, and, in general, rivets will not begin to work loose until the unit stresses are nearly twice as great as these.

Table 1 gives some of the unit stresses in bearing of the flange rivets of built-up stringers of railway bridges examined or computed by the writer. All these bridges were on railroads carrying heavy traffic, and, in computing the stresses, full allowance was made for impact, etc. Not one of these stringers, as far as the writer is aware, ever showed signs of failure. The tensile flange unit stress is also shown, by way of comparison with the bearing stress.

TABLE 1.—Stresses in Stringers and Floor-Beams of Railway Bridges.

Full allowance has been made for impact, etc., by the use of the impact formula, $I = L \frac{L}{L + D}$.

All these stresses were obtained from the wheel-loads actually used.

No.	Railroad.	Member.	No. of members.	Flange unit stress.	Maximum rivet-bearing on web.	Material.	Examined in:	Replaced in:	Remarks.
1	Railroad	Stringers		18 300	45 000	Iron.	1901		In use, 1905.
2*	"	"	32	10 200	39 000	"	"		" "
3*	"	"	4	9 500	52 400	"	"		" "
4	"	"	8	19 100	49 100	"	"		" "
5	"	"		15 500	38 000	Steel.	1900	1903–04	
6	"	"		27 900	31 000	"	"	"	
7	"	Deck-span		16 700	28 800	Iron.	"	"	
8	"	"		21 400	35 600	"	"	"	
9	"	"		23 500	37 000	"	"	"	
10	"	Floor-beams		16 000	38 000	"	"	"	
11	Traction Co.	"		23 300	33 100	Steel.	1903		In use, 1905.
12	"	"		23 050	38 300	"	"		" "
13	Railroad	Stringers		18 700	37 700	Iron.	1902		Not known, if in use now.
14	"	"		25 600	45 800	"	"		"
15	"	Floor-beams		20 600	43 000	"	"		"
16	"	Stringers	12	23 200	42 400	Steel.	1900	1904–05	
17	"	Floor-beams	5	21 000	35 700	"	"	"	
18	"	Stringers		19 520	36 700	"	"	"	
19	" Trunk line	"		29 800	52 300	"	1900	"	
20	"	Stringers and floor-beams	70	20 000 (av.)	40 000 (av.)	Iron.	"		Still in use, 1905.

* Nos. 2 and 3 were old stringers cut off and used as deck spans, which accounts for the great disproportion between the flange unit stresses and the bearing stresses of the flange rivets.

The writer has considered that the reason for this great discrepancy between theory and observed results lies in the neglect of the element of friction between the two faces of the members in contact. This theory seems to be confirmed by the results of such tests as have been made.

Tests to determine the friction of riveted joints were made at the Watertown Arsenal, in 1882. These demonstrated that the friction developed between the surfaces of the plates of a riveted joint is a function of the area of the rivet section, and, therefore, is directly comparable with the shearing value of the rivet. These tests show the value of the friction to be about 14 000 lb. per sq. in. of rivet section for the single surfaces in contact, for a lap-joint, and about 18 000 lb. per sq. in., for joints providing two contact surfaces. As these values are higher than those usually allowed for rivets in shear, it follows that, in riveted joints, the working stresses are transmitted entirely by friction, and the shearing and bearing resistances of the rivets are not brought into play until much higher values of stress are reached.

It would appear that the ordinary working values of rivets in shear and bearing might well be increased, in the design of new work, and that the maximum allowable stresses in old structures might also be increased.

European bridge engineers have stated that European practice allows a much higher value of rivets in shear and bearing than American practice, and that they have never had trouble from this source. Therefore, it is perfectly safe to allow shearing and bearing values equal to the allowed value of the material in tension for shear, and to double this amount for bearing, provided there are enough rivets in the connection, and that the workmanship is such as required to develop friction between the surfaces of the material. This would give values of, approximately, 22 500 lb. per sq. in. in shear, and 45 000 lb. per sq. in. in bearing, for rivet steel having a minimum elastic limit of 30 000 lb. per sq. in., to be used in examining old structures.

A set of experiments on riveted joints was recently made for a committee of the American Railway Engineering and Maintenance of Way Association.* These experiments were made on ninety

* Bulletin No. 62, American Railway Engineering and Maintenance of Way Association, April, 1905.

small riveted test pieces, which were tested to destruction, the failure occurring, in a majority of cases, by the shearing of the rivets at unit stresses of from 45 000 to 50 000 lb. per sq. in. of rivet section before driving, or by the breaking or splitting of the plates.

The values of the rivets in bearing upon the plates were approximately twice the shearing values at the point of failure. The conclusions drawn by the committee are repeated here:

"(1) That the resistance of a riveted joint against deformation by shearing forces, up to the yield point, is due to the friction between the surfaces held in contact by the rivets.

"(2) That the yield point of a riveted joint is reached when the shearing forces are equal to the friction of the surfaces held in contact by the rivets.

"(3) That the deformation of a riveted joint at the yield point is caused by the slipping on each other of the surfaces held in contact by the rivets, and is due to the diametral contraction of the rivets in cooling after they are driven, which leaves a space between the body of the rivet and the edge of the rivet hole.

"(4) That after the slip at the yield point has occurred and the rivet is brought to bear against the edge of the rivet hole, a deformation of the body of the rivet takes place with an accelerating increase in the resistance until the entire side of the rivet has been brought to bear against the edge of the rivet hole, and that the deformation continues beyond this point with a diminishing increase in the resistance until the ultimate strength of the rivet in shear has been reached and the breakdown occurs.

"(5) That lap joints, on account of the unsymmetrical distribution of material, deflect sideways under strain, throwing the rivets in tension, and thereby reducing the shearing forces between the surfaces held in contact by the rivets.

"(6) That fillers inserted between the main plates reduce the strength of a riveted joint, but that the full strength can be obtained by connecting the fillers to the main plates by additional rivets.

"(7) That the number of rivets connecting the fillers to the main plates should, for each intervening filler, be about one-third the number of rivets required in a similar joint without fillers, to obtain the same strength in both cases.

"(8) That the strength of a riveted joint with rivets of larger grip than about four times their diameter is decreased, as the length of the grip is increased.

"(9) That the number of rivets, in a riveted joint with larger grip of the rivets than four times their diameter, should be increased

at least one per cent. for each one-sixteenth of an inch increase in the grip above this length, to obtain the same strength as a similar joint with the grip of the rivets shorter than four times their diameter.

"(10) That a riveted joint, subject to forces always acting in the same direction, may safely be strained beyond the yield point up to a point where the rivets are brought to bear against the edges of the rivet holes.

"(11) That a riveted joint subject to forces alternating in opposite directions, may not safely be strained up to the yield point.

"(12) That, to obtain a minimum slip at the yield point, it is necessary that the holes in the component pieces should thoroughly match and that the driving tool should upset the rivet throughout its length so that it will thoroughly fill the rivet hole."

If the third conclusion be true, that rivets, in cooling, contract to such an extent that a space is left between the rivet surface and the side of the rivet hole, why is it that all rivets do not test loose under the inspector's hammer? It is the firm conviction of experienced inspectors that they can detect any rivet that is not completely upset. Is it possible that these men are mistaken, and that rivets which test loose do so only because the grip of the rivet heads on the plate is not great enough to prevent their movement?

This theory has been advanced before by European engineers, but the writer has seen specimens of riveted joints, which, when sawn through the rivet and polished, showed only a fine hair line dividing the rivet surface from the sides of the hole, which indicated a perfectly tight rivet. Of course, as is well known, in the case of long rivets, they are entirely upset only at the ends, and a space is often left between the rivet surface and the side of the hole at the center of the shank. Furthermore, the writer is loath to believe that the many thousands of rivets which he has tested and found to ring true could have done so unless the rivet sides were in contact with the sides of the hole, at least for some distance under the head.

Cannot the slipping of the plates, at the stress which is sufficient to overcome the friction between their surfaces, be explained on the ground of the compressibility of the metal, since apparently the slipping always occurs at a high stress. The experiments of M. Considère, in France, indicate slipping at a stress equal to ap-

proximately four-tenths of the ultimate shearing strength of the rivets, while the records of the experiments, conducted by the committee referred to, do not indicate any extraordinary movement of the plates when the slip occurred, subsequent increments of load causing movements of the plate comparable to the original slip. Is it not probable that the driving operation causes a state of compression to exist in the rivet metal and in the surrounding plate sufficient to take up the contraction due to cooling?

When the stress becomes sufficient to overcome the friction between the plates, the sudden reduction of the coefficient of friction would bring about half this stress suddenly upon the rivets, and considerable movement would be expected, due to the compression of the material, even if the rivets were perfectly tight.

There is another point, which concerns the design of new work as well as the investigation of old work, to which attention should be called.

In the design of plate girders it is the general custom, in the United States, to take the effective depth of the girder as the distance between the centers of gravity of the flanges, when obtaining flange stresses, and the distance between the rivet lines as the effective depth when obtaining the rivet values, which are dependent upon the stress transferred from the webs into the flanges in a given distance. Since the latter stress is simply an increment of the former, it would seem inconsistent to use different depths of girder for the two cases. Would not the more rational method be to assume the effective depth of the girder as variable, and use the same depth for both purposes?

At the ends of the girder there is no flange stress, and, therefore, the effective depth of the girder is evidently the distance from center to center of rivets; while, at the center of the girder, considering the same to be uniformly loaded, there is no shear, and all the stress has found its way into the flanges. Evidently, then, the distance between the gravity lines of the flanges is the correct depth of the girder at this point, if the stress be uniformly distributed throughout the flange section, as it seems reasonable to assume.

The writer has found many cases of existing girders in which the maximum rivet stresses were found, not at the ends of the girder, but at an intermediate point; and he would recommend that,

in such cases, a depth intermediate between the distance from center to center of gravity of the flanges, and the distance from center to center of the rivet lines, be taken as the effective depth for all purposes.

There is another point which illustrates well the difference between rules for designing new structures, and those for investigating old ones.

It is common practice, in designing plate girders, to allow no part of the web to be considered as flange section. While this gives an added factor of safety to the structure, and may be commendable in designing new work, there can be no doubt that the web will do its share of the work involved in resisting flange stresses, in spite of the prohibition, and this fact should be fully considered in determining the strength of an existing structure, as, in such cases, it is desirable to know the actual conditions of stress.

The writer is a firm believer in the use of standard specifications as a guide in designing standard structures, and believes just as firmly that they should not be followed in designing structures which are of an extraordinary or unusual nature, and should not be used when reporting upon the safety of a structure, which is, in itself, unusual, or is subjected to unusual conditions of loading.

The general principles of bridge design should constitute the engineer's sole guide in such cases, and his only purpose should be to determine the actual conditions of stress in the various parts of the structure, basing his computations on actual measurements, and "rock-bottom" principles, unhampered by the limitations and assumptions of a set of specifications gotten up for an entirely different purpose.

DISCUSSION.

R. D. Coombs, Assoc. M. Am. Soc. C. E. (by letter).—The writer hopes that the discussion of this paper will bring out the various methods used in treating old railroad bridges, particularly with regard to unit stresses and impact allowances. Mr. Coombs.

It does not seem to be consistent, when figuring new work, to claim that the "300 formula," for instance, represents closely the actual conditions, and to refigure old bridges by a reduced impact.

Unless a "stop order" is issued, the impact load exists equally for old and new structures, assuming no excessive vibration or deflection in the former. The variation will then appear in the unit stresses, the question being the relation of the existing stress to the elastic limit. It would be interesting to know whether the rigid adherents of the "300 formula" request a "stop order" at bridges whenever the unit stresses amount to 75% of the elastic limit. The writer's preference is for the following method, using the impact curve marked *A* in Fig. 1:

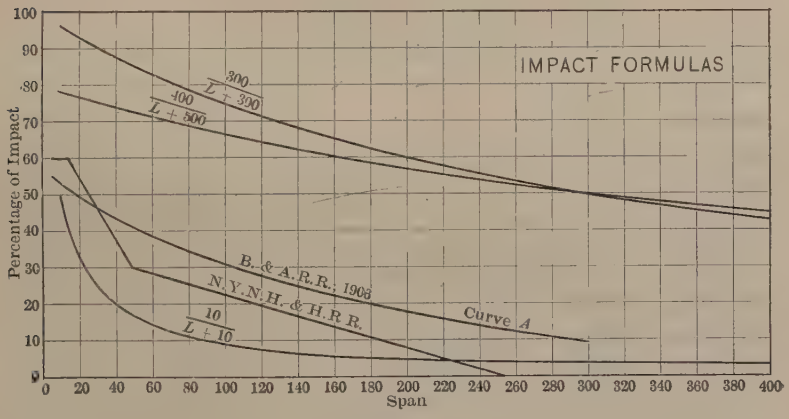


Fig. 1.

First.—Calculate the existing stresses from the drawings, when obtainable, otherwise take field measurements of the structure.

Second.—Examine the structure for corrosion, net area, loose rivets, wear at joints, excessive vibration or deflection, and reversed or eccentric loading, paying especial attention to points found when figuring from the drawings.

Third.—Revise the figured stresses, if necessary, as a result of the field inspection, and then examine the results with reference to the figures in Table 2.

Mr. Coombs.

TABLE 2.—MAXIMUM UNITS FOR OLD STRUCTURES.

(The figures are for both dead and live loads.)

Kind of material.	Elastic limit.	Tension and compression. $0.80 \times$ E. L.	Shear. $\frac{2}{3} \times 0.80 \times$ E. L.	Bending. $\frac{4}{5} \times 0.60 \times$ E. L.	Bending pins. $\frac{4}{5} \times 0.80 \times$ E. L.	Shop Rivets. Shear. $\frac{2}{3} \times$ E. L.	Shop Rivets. Bearing. $\frac{4}{5} \times$ E. L.	Field Rivets. Shear. $\frac{2}{3} \times 0.75 \times$ E. L.	Field Rivets. Bearing. $\frac{4}{5} \times 0.75 \times$ E. L.	Rollers.
60 000 to 68 000	33 000	26 500	17 600	27 000	35 200	.....	.....	.....	.....	1 000 D.
56 000 to 64 000	30 000	24 000	16 000	24 000	32 000	.....	.....	.....	.....	900 D.
Soft steel	25 000	.....	.....	.....	.....	16 650	33 300	12 500	25 000	.....
Wrought iron.	20 000	.....	.....	.....	.....	13 300	26 600	10 000	20 000	.....

Regarding the author's recommendation as to the effective depth of plate girders, the writer is of the opinion that ordinarily it will make little practical difference which effective depth is used.

Mr. Merriman.

MANSFIELD MERRIMAN, M. AM. SOC. C. E.—This paper contains sound doctrine, and calls attention to a subject which is becoming of greater importance every year. Numerous highway bridges are now overloaded, owing to the addition of sidewalks and trolley tracks, so that the stresses in the members are much higher than intended by the designers. The speaker has in mind a wrought-iron bridge, built about thirty years ago, with one footwalk, the truss on that side being made heavier than the other one. A second footwalk and a trolley track have been added on the same side as the lighter truss, so that the load brought upon it is about 60% heavier than that intended, while the floor system is so highly stressed as to be likely to fail under full load.

The concluding paragraph of the paper deserves especial notice, as giving principles which always should be observed, but are often violated. Under no circumstances can it be allowed, in investigating a bridge, to use the drawings which were prepared by the designer or manufacturer, for it cannot be assumed that the bridge was built in conformity with those drawings. Specifications for the design of bridges have also little applicability in deciding whether or not an existing structure should be condemned. Even in repairing or strengthening an old bridge, rules and specifications are materially different from those used in design, especially when the repairs are only intended to lengthen the life of the bridge for two or three years.

Mr. Merriman.

The author makes no reference to pins, but it will often be found that they are very highly stressed in old highway bridges to which a trolley track has been added. It is the speaker's opinion that higher bearing and flexural stresses may be permitted on these pins than in any other part of the truss, since the failure of a pin appears to be very rare. It should be noted, however, that there must be a considerable distortion by bending in such pins, and that this affects the distribution of the stresses in the eye-bars whenever there are more than two bars in one panel, so that some of the eye-bars may receive stresses much higher than they are supposed to have, under the usual assumptions.

Mr. Seaman.

HENRY B. SEAMAN, M. AM. SOC. C. E.—The key to the rational investigation of overloaded bridges would be a formula for the dynamic increase of live-load strains which will give a strain equivalent in effects to that of quiescent or dead load. Such a formula has not yet been discovered, and probably never will be; but, if such could be obtained, it would seem to be entirely safe to permit 20 000 lb. per sq. in. on structural steel of 60 000 lb. ultimate strength, and when the bridge is under constant supervision it would seem to be permissible to increase this to, say, 24 000 lb. per sq. in., the allowable strains on wrought iron and on a higher steel to be modified accordingly.

The author refers to the formula, $I = L \frac{L}{L + D}$, as a rational impact formula, but does not explain its rational deduction. Any formula containing D (dead load) would not seem to be an impact formula, although it might be an empirical formula for dynamic increase, which would include impact, oscillation, fatigue, "ignornance," etc. As far as the speaker has been able to ascertain, the proper form of an impact formula should be: $I = \frac{C'}{\sqrt{L}}$, in which L is the distance traveled by the load, in producing the given strain, and equals approximately one-half the span; and he was advised some years ago by B. W. Guppy, M. Am. Soc. C. E., that a value of $\frac{9}{10}$ for the constant C' would conform approximately to the experiments made by E. Herbert Stone, M. Am. Soc. C. E. As this formula is based upon the assumption that the strain produced by a load instantaneously applied is double that of the same load at rest, and also that the effect of a suddenly applied load varies inversely as the time consumed in its application, the value of I would be 100% for a very short span, and would thence decrease to zero for a span of 0 ft., provided the speed of the train was constant. The impact for the shortest spans would not be less than 100%, however, as the speed may decrease.

If this formula for impact were enlarged to cover the entire

Mr. Seaman. dynamic increase of an applied load, another constant, C'', should be added, making the formula: $I = \frac{C'}{\sqrt{L}} + C''$.

If this constant is intended to cover the effect of oscillation, its value would be replaced at from 30 to 50%, varying according to different authorities.* If it were also intended to cover the fatigue, "ignorance," etc., it would probably be further increased.

To the speaker's mind, such a modification of live-load strains is desirable, not merely for the investigation of old bridges, but also for the design of new bridges, except that, for old bridges, which receive constant supervision, the value of C'' might be less than would seem to be advisable in a new structure, subject to no special supervision, and to a possible increase of loading. The advantage of this method of design would be to produce heavy counters, and thus prepare for the increased weight of traffic which experience has shown may be expected.

The author suggests that more failures occur in compression members than in tension members; but, if such is the case, it would seem to be due to improper bracing, or to improper design of details, or possibly to accident, rather than to insufficient theoretical cross-section.

He mentions the usual assumption of the shearing strength of material as equal to two-thirds of the tensile strength, and suggests that this is too small. The speaker usually assumes the shearing strength of material as equal to three-fourths of the tensile strength, and the bearing strength as equal to one and one-half times the tensile strength. If, however, allowance is made for softer rivet steel, the corresponding modification might be made in the allowable strains.

The general subject of the investigation of overloaded bridges is an interesting one, and accumulated data and comparison of views are always of value. The theoretical analysis of a structure should be thoroughly made, and will aid, though it cannot replace, individual judgment.

A number of years ago, in the early days of iron bridges, the speaker was called upon to examine a deck-truss, revolving drawbridge, reported to be in dangerous condition. He examined the drawings and records of the structure, and learned that the bridge had already been condemned twice by different engineers. In examining the structure it was found to have been built in accordance with the drawings, but to have had two accidents, due to vessels striking the bridge when closed. The fractures thus caused were carefully and properly spliced, but the main tension diagonals were so slack that they were spread apart by blocks of

* *Transactions*, Am. Soc. C. E., Vol. XLI, p. 240.

wood in order to bring them in tension. The counter diagonals were tight, and apparently doing service. Recalculation of the bridge showed that the sections were sufficient for the loading, and the speaker ventured to suggest that the bridge needed readjustment. Mr. Seaman.

A roadmaster and track gang were placed at his disposal, the wooden blocks were knocked out, and the men were instructed to unscrew the counter rods. When one of these rods was unscrewed 1½ in. the men refused to do further work, fearing that the bridge would fall. The foreman was thereupon called to the center pier and told to look over the new line of the chord. He was amazed and delighted, and the further adjustment continued without objection. The bridge was thereafter reported in first-class condition until replaced several years later by a double-track structure.

This instance is cited merely to show the advisability of careful investigation and thoughtful study as to causes of deformations and apparent weakness, and also as an assurance that reliance should be placed upon theoretical deductions, carefully made.

GEORGE B. FRANCIS, M. AM. SOC. C. E.—About five years ago the speaker was Chief Engineer of a street railway having 270 miles of track, operating in eighteen different communities. At the time of accepting the engagement, one of the first things the speaker tried to ascertain was the condition of the various bridges over which the road was operated. Mr. Francis.

Among the papers of the company no very complete record of the condition, characteristics, or systematic inspection of these bridges could be found, and it might interest the members to know what was done in getting together some useful data and establishing a method of inspection.

There were about 135 bridges: Some had been built for streets and highways by the various municipalities and towns; some by existing steam railroads, over their tracks on which the highways and street railways were located; some built on private right of way by the street railway company; and some built on highways by the street railway company. These embraced all classes of structures.

One of the first things done was to make a list showing their general characteristics, the date they were built, who built them, who was the owner, and who was under obligation to strengthen, renew, or repair them.

Then a frequent and practical examination was instituted, and a record made, so that, in case of trouble, it would be possible to go before a court and testify that the bridges were being looked after in a systematic way.

A good many of these bridges were strengthened in consequence of this examination. On one of the routes, a regular steam railroad freight car service was maintained, the freight cars being hauled by

Mr. Francis. electric locomotives. The bridges on this line were built or strengthened for corresponding loads. On other lines there was the usual street car equipment and also a heavy type of electric interurban car. This latter type was the one considered for loads in making calculations for strains. If the tensile strains in the iron or steel did not exceed 20 000 lb. per sq. in., the bridges were considered safe; if the strains were greater, the bridges were strengthened. In most cases the greater strains occurred in the floor stringers, and various expedients were adopted to insert new members. Generally, the steel bridges were found to be strong enough, except as stated.

A good many minor structures of wood and stone were in all stages of repair, and were quite easily strengthened. There was considerable trouble with the various owners as to the division of expenses and the manner of doing the work. As a rule, the expense of strengthening fell upon the street railway company.

Mr. Belzner. THEODORE BELZNER, JUN. AM. SOC. C. E. (by letter).—The writer agrees with the author that "it is the firm conviction of experienced inspectors that they can detect any rivet that is not completely upset."

The "space between the body of the rivet and the edge of the rivet hole," due to the contraction of the rivet in cooling, is more likely to occur with hand-driven than with machine-driven rivets. Assuming that the rivets in the tests mentioned by the author were hand-driven, there is no doubt that such a space might be left between the rivet surface and the sides of the hole, because the upsetting of hand-driven rivets seldom, if ever, fills the holes completely. On the other hand, when rivets are machine-driven, the heads are perfectly formed and the shank is upset so that it fills the hole completely.

During the construction of several large viaducts in New York City, the writer made many tests and observations of machine-riveted conections, and generally found only a fine hair line dividing the rivet surface from the sides of the hole, which indicated perfectly tight rivets.

The case of long rivets recalls to the writer an examination he made of a riveted connection for a very important bridge member. The bent plate of this member contained 87 long 1-in. shop rivets, 28 of which were condemned for looseness. The shop rivets were cut out and replaced by machine-driven field rivets. In cutting off the heads and "backing out" the rivets, no material damage was done to the adjacent ones, but many of them became loosened while the field rivets were being driven. The rivets "backed out" easily, some requiring only a few blows with an 8-lb. hammer. An examination of these rivets showed that they had not been upset properly, in some cases shoulders were formed under the heads (caused by

oval-shaped holes through the bent plate), in other cases the rivets were slightly rusted and showed some scale. The result was that more than 60 field rivets had to be driven to replace the shop rivets. Mr. Belzner.

J. E. GREINER, M. AM. SOC. C. E. (by letter).—At the Annual Convention of the Society in 1895, the writer presented a paper entitled "What Is the Life of an Iron Railroad Bridge?"* which covered precisely the same ground as the one now under discussion. Mr. Greiner.

The extent to which a bridge could be overloaded, as recommended by the writer at that time, was very fully discussed, and the opinion of those taking part in the discussion quite generally agreed with that which he advanced.

Mr. Watson's limits are based on what he calls a "rational impact formula," which differs somewhat from that used formerly by the writer, and also differs from the impact formula adopted by the American Railway Engineering and Maintenance of Way Association, now quite commonly used.

Mr. Watson's limits as to the extent to which a bridge can be overloaded are conservative and somewhat lower than those which were advocated by the writer in 1895 and have been his practice since that time.

WILBUR J. WATSON, M. AM. SOC. C. E. (by letter).—The comparative diagram of impact formulas given by Mr. R. D. Coombs, is very interesting. The writer has compared the results given by the "300 formula," with those given by the formula which he uses and calls the "rational formula," and has found that these results differ very little from each other in all ordinary cases. He prefers the "rational formula" because, to his mind, it is rational. Mr. Watson.

Mr. Seaman, in criticising the use of this term, appears to have in mind the strict mechanical meaning of the term "impact," which is defined by Webster as "the single blow or stroke of a body in motion against another, either in motion or at rest," but the impact formula under discussion is intended to cover, not impact itself—which is a function of the properties of the striking body only—but the effects of the impact upon the body struck, which is an entirely different matter, and does involve the physical properties of the latter body.

The amount of impact has already been assumed in this formula, as well as in the "Launhardt" and "300" formulas, to be equal to the live load; but the effect of this impact upon the body struck is a function of its elasticity and mass, the latter body being always at rest. The writer did not intend that this formula should be taken as a measure of impact, but as a measure of the stress due to impact, and the word "stress" should be understood as following all the initials, *I*, *D* and *L*, which enter into this formula. Aside from

* *Transactions*, Am. Soc. C. E., Vol. XXXIV, p. 294.

Mr. Watson. theoretical analysis, it appears to the writer that anyone who has carefully observed the effects of impact upon bridge structures must be convinced that the dead weight of the structure plays a very important part in reducing the effect of the impact due to an applied load.

The writer confesses that he had overlooked the very valuable paper by Mr. J. E. Greiner, entitled "What Is the Life of an Iron Railroad Bridge?" which covers much of the ground now under discussion, and wishes to commend this paper to the younger members of the profession. He can see no reason for changing the limits of stress, which he has proposed, but this discussion and the reading of Mr. Greiner's paper, together with the valuable discussions thereon, especially that by Mr. Schneider, have convinced him that these limits are conservative.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS

Paper No. 1035.

FOUNTAIN FLOW OF WATER IN VERTICAL PIPES.*

By Messrs. F. E. Lawrence and P. L. Braunworth.

With Discussion by Messrs. A. Flamant, Edgar C. Thrupp, Frank L. Getman, Ernest W. Schoder, F. E. Lawrence and P. L. Braunworth.

Introductory.

The purpose of the experiments described in this paper was to obtain a general law for the fountain flow of water from vertical pipes, for pipes of any size and for any head over the crest.

It has long been an accepted theory, supported by demonstration in physical laboratories, that the height of a jet of water equals nearly $\frac{V^2}{2g}$, the difference being supposed to be due to air resistance; but the theory does not take into consideration the weight of the column of water and the fact that the condition of flow for high heads is entirely different from that for low heads.

For very low heads the flow is almost identically the same as the flow over a sharp-crested weir. In both cases, the water flows over the edge in a smooth sheet. If there is a vacuum beneath the sheet, it clings to the weir in both cases; and if air is admitted, the sheet jumps free.

* Presented at the meeting of October 17th, 1906.

In passing from the quiet sheet flow to the jet, the flow is of a peculiar character. It begins with a swirling or rotary motion, and ends in a bubbling condition. Just when the jet is finally formed cannot be detected by the eye, as the water in falling hides the jet column.

The results of the experiments to be described give definite information, which confirms very closely the theory of jet flow for high heads, and indicates a law that differs materially from this theory for low heads. Also, the nature of the transition from one condition to the other has been clearly defined, and definite laws have been determined giving, for pipes of any size, the head at which this intermediate stage begins and ends.

That these laws of flow are of considerable importance is patent to all who have tried to determine the flow from fountain influents to filter plants, or from Artesian wells. The flow in these cases is most often in the condition where the jet formula does not apply.

As far as the writers know, there has been no previous attempt to determine these laws of flow by experiment, so that they would be generally applicable to pipes of any size. Professor G. E. Todd, State Geologist of South Dakota, has computed a table* on the theoretical jet formula, which does not take into consideration the weir and intermediate conditions. As will appear later, the use of this formula involves very great errors for low heads, giving entirely misleading results, and making the formula worse than useless for these cases.

The United States Geological Survey has for some time tried to obtain experimental data in order to establish these laws of flow, and in the spring of 1905 corresponded with E. W. Schoder, Jun. Am. Soc. C. E., in charge of the Experimental Hydraulic Department of Cornell University, regarding the performance of such experiments. Shortly afterward Messrs. Elliot and Porter took up the matter for a graduating thesis; but, on account of the small head of water obtainable in the basement of the civil engineering building, only the weir condition on one pipe (4-in.) was investigated. One valuable result, however, was brought out in this experiment. It was found that a pressure tap in the side of the vertical pipe was

* "The Motions of Underground Waters," by Charles S. Slichter, U. S. Geological Survey, Water Supply and Irrigation Paper No. 67.

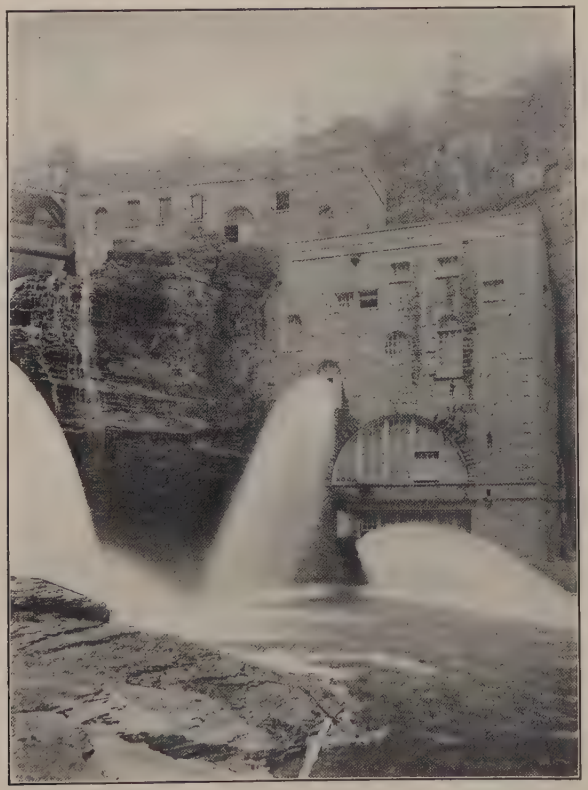


FIG. 1.—HYDRAULIC LABORATORY OF CORNELL UNIVERSITY.

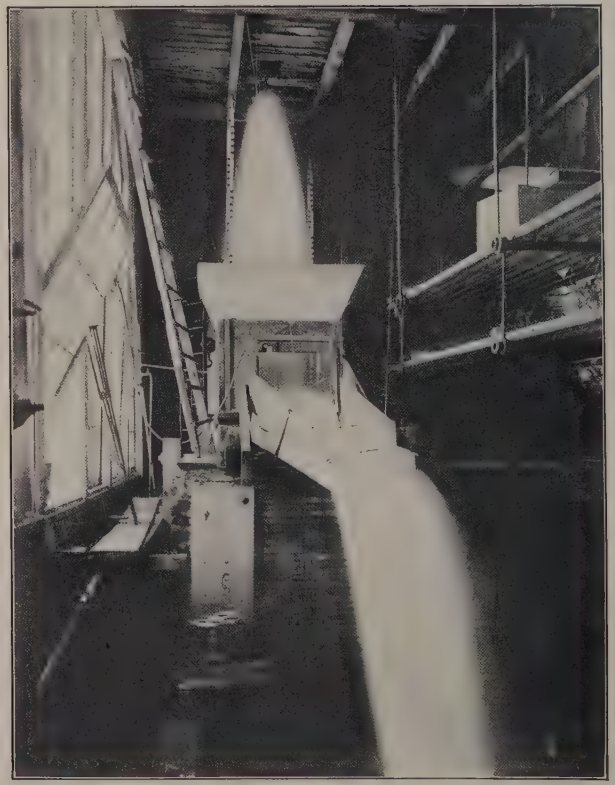


FIG. 2.—APPARATUS DURING A RUN ON A HIGH HEAD, WITH A 4-IN. PIPE.

entirely inadequate for indicating heads. A Pitot tube gave far better results.

In the early spring of 1905, the writers determined to spend the succeeding summer in obtaining experimental data on this subject, using as wide a range of sizes of pipes as possible.

The College of Civil Engineering generously offered to supply the greater part of the equipment needed in the work, and Professor Schoder arranged to make use of the peculiar advantages offered by the Hydraulic Laboratory in Fall Creek Gorge.

The writers feel deeply indebted to Professor Schoder for his valuable advice, and skilful planning and arrangement of the apparatus used. It is to his encouragement and enthusiasm that the successful issue of these experiments is largely due.

The experiments were begun in July and finished in September, 1905, covering a period of seven weeks.

Description of Apparatus.

Hydraulic Laboratory.—The Hydraulic Laboratory of Cornell University is situated in Fall Creek Gorge just below Trip-Hammer Falls. The building encloses a stand-pipe, 6 ft. in diameter and 68 ft. high, which is used as a supply for experimental purposes on the various floors of the building, and receives its supply, either from a 30-in. pipe or from the 16-ft. experimental canal. Both these sources are fed from Beebe Lake, which is just above the falls. The supply is led through baffle boards into a weir chamber, 25 ft. long, and over a sharp-crested weir, into the stand-pipe.

Fig. 1, Plate XXIII, is a view of the building, and the general elevation of the stand-pipe and experimental apparatus is shown by Fig. 5. In Fig. 1, Plate XXIII, may be seen the discharge from the 12-in. regulating gate on the second floor, and also from the 36-in. gate at the bottom. The latter gate is operated by a hydraulic lift, and was used to empty the stand-pipe at the end of the day's work.

Apparatus.—The apparatus common to all experiments, Fig. 5, consisted of a 12-in. vertical pipe connected at the bottom to a stand-pipe nozzle by an elbow and a gate-valve. This pipe sup-

ported a large cypress box having a spout. It was arranged so that the flow from the box could fall into the large lead-lined measuring tank, or, by means of a galvanized-iron diverter, could be led clear over the side of the tank and let fall to the concrete floor below, where several drains carried it to the creek.

The 12-in. pipe was of ordinary well-casing, and was fitted at the lower end with a plain flange for bolting to the elbow, and at the upper end with a special flange, which secured water-tightness and at the same time helped to support the box. The box was provided with a flaring galvanized-iron top-piece, 1 ft. 7 in. high. This was fitted into the top of the box, and served to prevent loss of water during high-head runs.

The trough diverter was provided with extension sides in order to carry off the higher heads and also prevent splashing. The upper end was given a slope of 45° so as to form a good cutting edge when pushed through the stream of water. The latter arrangement made it possible to cut off the supply instantly. The diverter in operation, wasting the flow from the 4-in. fountain, with a head of about 8 ft., is shown in Fig. 2, Plate XXIII.

Arrangement of Pipes.—The pipes used in these experiments were 12, 9, 6, 4 and 2-in., giving, in the writers' opinion, a sufficiently wide range. It was also thought that the experiments could be made more valuable by using pipes of the sizes used in actual practice.

TABLE I.—Dimensions of Pipes.

Length, in Feet.		Diameter, in Inches.		Thickness, in inches.	Position of Pitot tube insertion below top, in inches.
Weir part, in feet.	Stem part, in feet.	Nominal.	Measured and accepted.		
2.70	6.00	2	2.080	0.10	1.90
3.10	9.65	4	4.090	0.14	4.00
3.05	10.85	6	6.197	0.19	6.20
2.83	11.90	9	9.172	0.22	9.20
2.99	12.00	12	12.586	0.16	12.66

In the descriptions following, the nominal, instead of the actual, diameters are used.

The total length of these pipes varied from 12 to 15 ft., which length helped to restore the abnormal conditions of flow, caused

ARRANGEMENT OF PIPES FOR FOUNTAIN-FLOW EXPERIMENTS

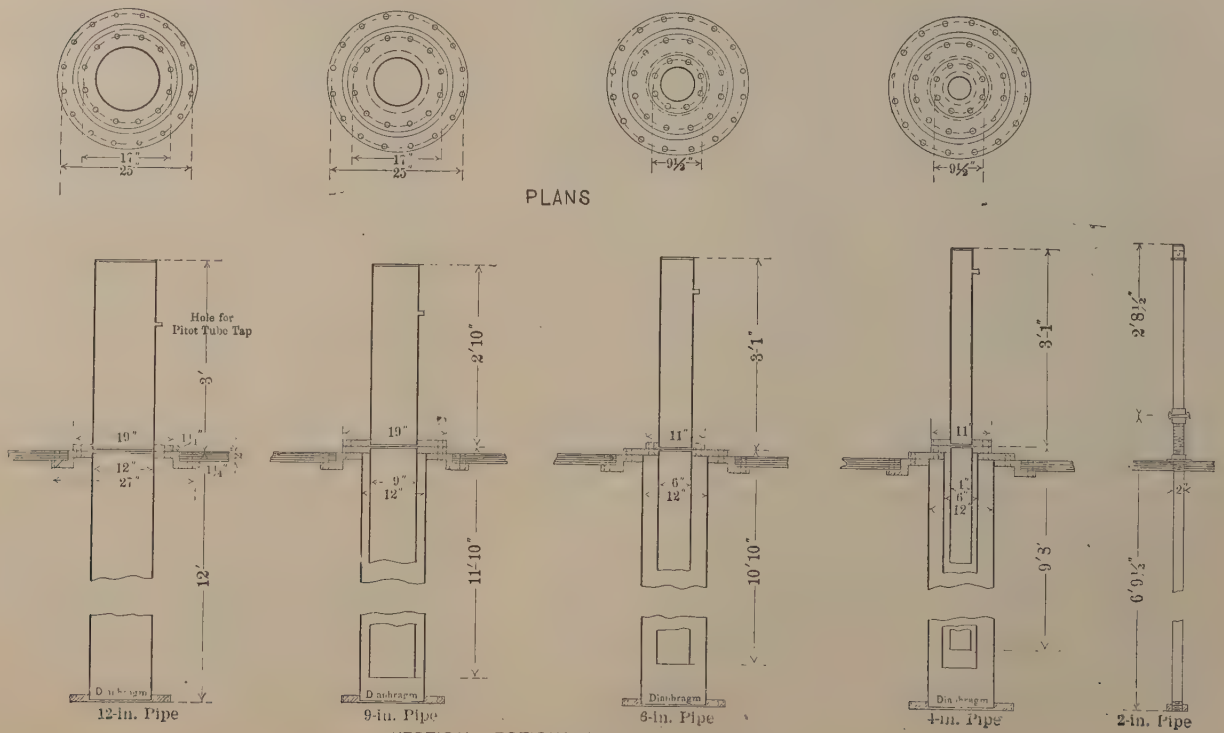


FIG. 1.

by the elbow, to more nearly normal conditions, at the top. The 12-in. pipe was of well-casing, and the 9, 6 and 4-in. pipe of standard wrought iron. The 2-in. pipe was of seamless drawn brass.

Fig. 1 shows the arrangement of the pipes. Each pipe was composed of two parts, the upper or weir length, and the lower or stem. The stems had flanges so as to fit the large flange of the 12-in. pipe when lowered into place. This arrangement was made for the convenience of leaving the stems in place, which was done in all cases except for the 9-in. pipe. The weir lengths and their flanges were the only parts removed. The tops of these lengths were turned true in a lathe, both inside and outside, to a depth of about 6 in., and the upper edge was beveled off on the outside at an angle of 60° with the horizontal, thus giving in effect a sharp-crested weir. At a distance from the top, equal to the diameter of the pipe, a tap was made for the attachment of the Pitot tube sheath.

After cleaning off the rust, four measurements of diameters were made with steel calipers near the upper edge of the weir.

The 2-in. combination was fitted up after the experiments had been made with all the other pipes. For this the weir length of the 4-in. pipe was removed and a blank flange put over the top of the 4-in. stem. Then the 2-in. stem was fitted to the elbow from the 6-in. gate on the stand-pipe (Fig. 5). This stem, when put through a hole in the box, projected a short distance above the floor. A collar, serving as a lock-nut, was then screwed down over the stem. This prevented leakage. The 2-in. weir length was made up of two pieces, the top being made specially for the purpose.

In order that the experiments might be made under conditions of flow as nearly normal as possible, the effect of the 12-in. elbow had to be compensated. This was done with a galvanized-iron diaphragm, placed between the top flange of the elbow and the bottom flange of the 12-in. pipe. This was arranged to provide resistance to the water leaving the convex side of the elbow, and free flow to the slower moving water from the concave side (Fig. 2).

It was thought that this would produce nearly normal flow above the elbow. That this proved to be correct is shown by

PLATE XXIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVII, No. 1035.
LAWRENCE AND BRAUNWORTH ON
FOUNTAIN FLOW OF WATER.

FIG. 1.—HIGH JET FROM 2-IN. PIPE.

FIG. 2.—WEIR CONDITION ON 12-IN. PIPE.

Plates XXV and XXVI, the velocity traverses taken by the Pitot tube.

Pitot Tube.—The Pitot tube used for measuring heads above the crest of the pipe, is shown in section with the sheath in Fig. 3.

The tube proper consists of a straight cylindrical brass tube, having an external diameter of 0.17 in. and an internal diameter of 0.10 in., being 9.6 in. long, and closed at the lower end. The

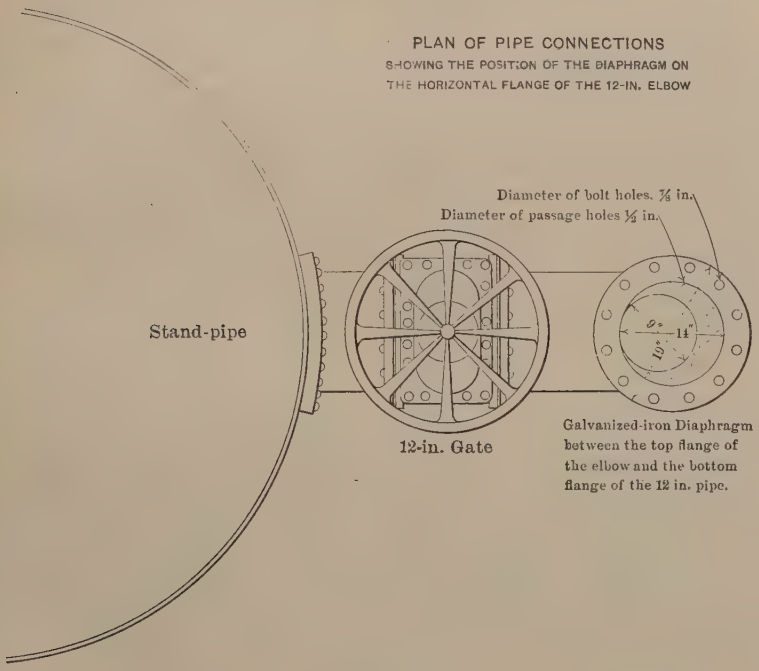

PLAN OF PIPE CONNECTIONS
SHOWING THE POSITION OF THE DIAPHRAGM ON
THE HORIZONTAL FLANGE OF THE 12-IN. ELBOW
Diameter of bolt holes. ⅞ in.
Diameter of passage holes ½ in.
Stand-pipe
12-in. Gate
Galvanized-iron Diaphragm between the top flange of the elbow and the bottom flange of the 12 in. pipe.

FIG. 2.

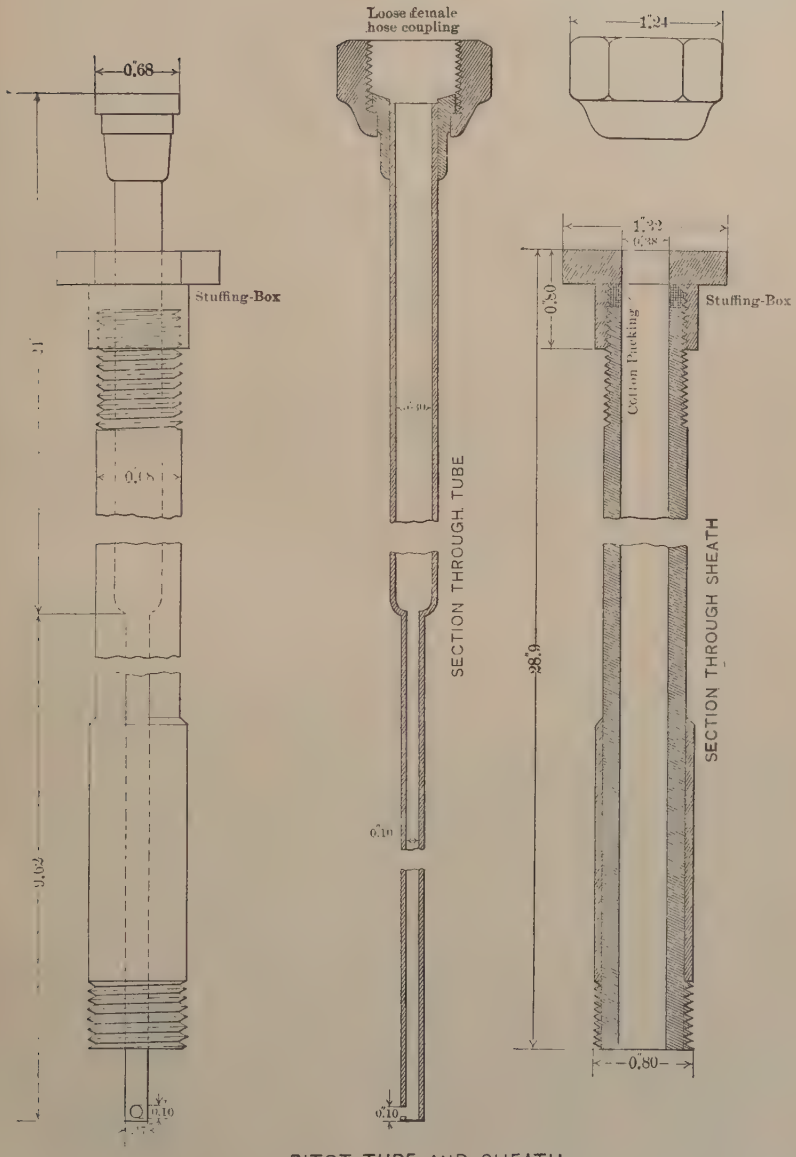
impact opening is a hole, 0.10 in. in diameter, drilled at right angles to the axis of the tube and 0.10 in. from the end. At the outer end the tube widens out to a diameter of 0.30 in., so as to slide snugly through the stuffing-box at the end of the sheath. Near the end of the tube is a pointer for indicating the direction of the impact opening. The sheath screws into the hole drilled through the pipe wall. The burr inside the pipe, made by the drill, was carefully filed off. When the sheath was screwed into the hole,

care was taken that it did not extend into the pipe. The other end of the tube has a female coupling, to which the hose was attached. The hose was connected with one arm of a glass gauge, mounted on a wooden frame, on which was a scale graduated in feet and hundredths. By opening the stop-cock at the top of the gauge, the arm was used as an open column. As the gauge was only 2 ft. high, it had to be raised when the head exceeded this amount. Several changes were necessary for the smaller pipes.

Sighting Rods.—In order that the results of these experiments might be put into use in the field, where the Pitot tube is not always available, sighting rods were used. These were ordinary stadia rods, blocked off in red and white to tenths of feet. They were placed vertically in opposite corners of the box, so that a sight might be taken over the center of the pipe. They were leveled up, so as to give equal readings for the same elevation. The sighting rods were not used until after the runs on the 9-in. pipe had been made.

Measuring Tank.—The measuring tank is of 4-in. pine, and is lined with sheet lead. Its inside dimensions are: 15 ft. 10 in. by 10 ft. by 5 ft. 5 in., and its capacity is about 850 cu. ft. Two tape blocks and boxes were fastened securely to the top of the tank on opposite sides. Steel tapes carrying long-pointed plumb-bobs were run over the blocks and fastened to spools in the boxes. These tapes were graduated in feet and hundredths. On the tops of the blocks were brass plates, grooved to allow the tapes to slide through. Readings were made on the tapes opposite reference scratches on either side of the groove.

Rating.—In order to measure the quantity of water during a run, the measuring tank was rated in the following manner: The readings on the tape for the bottom of the tank were first taken. Then 5 cu. ft. of water was weighed out in a galvanized-iron tank on a small Fairbanks scale, and emptied into the large measuring tank. When five of these had been poured in and the water quieted, the tapes were read again. Readings were taken by lowering the plumb-bobs until their points just touched the water surface. For the remainder of the rating, readings were made for every 50 cu. ft. Plottings were then made showing the relation be-



PITOT TUBE AND SHEATH

Fig. 3.

tween the tape readings and the quantities for each tape. This was practically a straight line in each case. Then, if the quantity for any run was required, it could be obtained by picking off the quantities for the initial and final tape readings. The difference represented the quantity for the run.

Experimentation.

The readings on the Pitot-tube gauge, corresponding to zero head on the crest, were found in the following manner: As shown

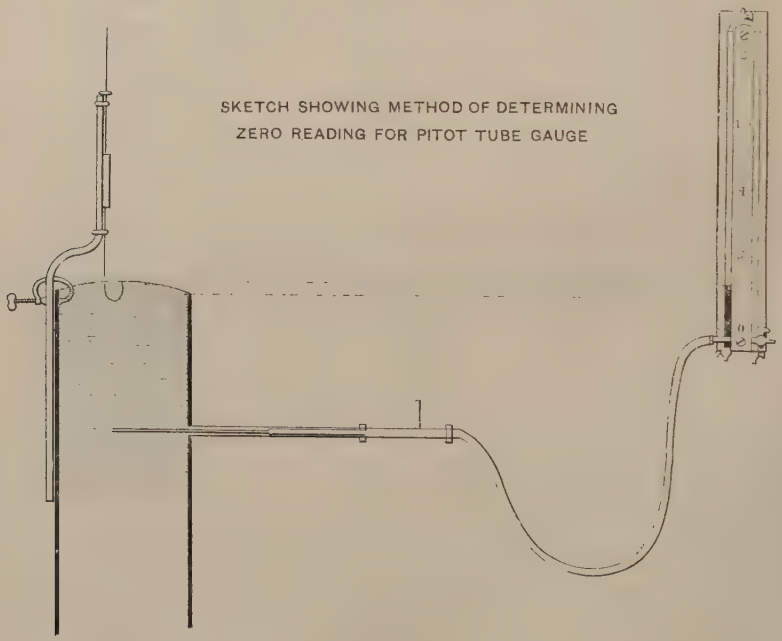
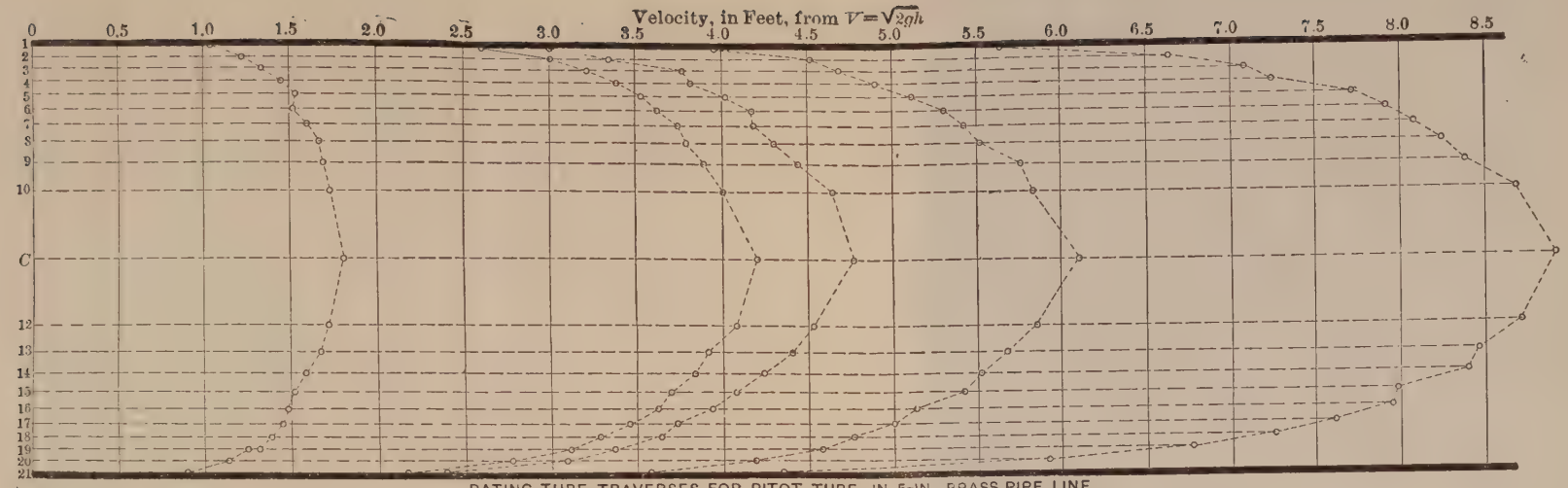
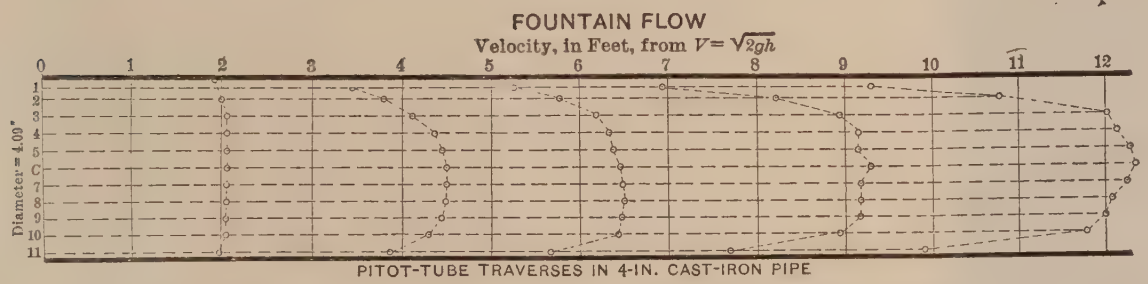


Fig. 4.

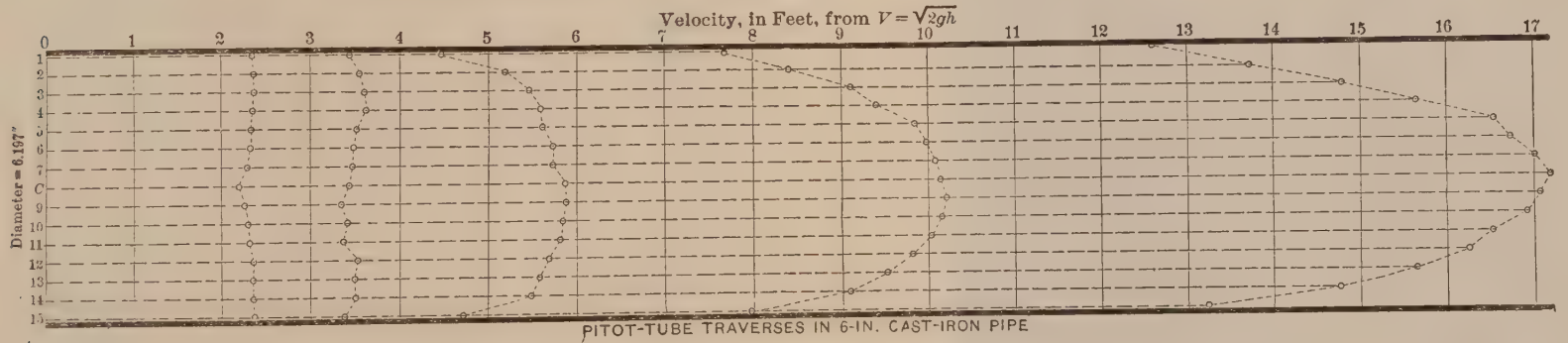
in the sketch, Fig. 4, a hook-gauge, reading to 0.0001 ft., was clamped to the side of the pipe. The point of the gauge was brought to the exact level of the crest by supporting one end of a small spirit-level on the point of the hook-gauge, with the other end resting on the edge of the pipe, along a diameter which was previously found to be level. The readings were made when the



RATING-TUBE TRAVERSES FOR PITOT TUBE, IN 5-IN. BRASS PIPE LINE



PITOT-TUBE TRAVERSES IN 4-IN. CAST-IRON PIPE



PITOT-TUBE TRAVERSES IN 6-IN. CAST-IRON PIPE

bubble came to the center. Water was then let into the pipe until it was full. Then the gate was closed, but, owing to a small leakage in the valve, there was a very slight overflow. Several settings for the water surface level were made with the hook-gauge, and simultaneous readings were taken on the glass-gauge. The glass-gauge was read by placing the upper edge of a small brass runner tangent to the bottom of the meniscus of the water column, and reading on the scale alongside. The difference of the average of the two sets of hook-gauge readings, subtracted from the average of the glass-gauge readings, gave the zero reading of the glass-gauge referred to the crest level. This determination of the zero reading was made several times for each pipe, because the deduced heads depend largely upon it for their accuracy.

It was of considerable importance to keep the hose connection of the Pitot tube entirely free from air. When air gets into the tube, the readings on the gauge are either too high or too low, but usually the latter. To prevent air from accumulating in the bends, the hose was frequently shaken out. By sucking on one opening of the gauge a siphon was formed and the air brought out. In one case a small hand-pump was used to good advantage.

In order to prevent the pounding of water in the stand-pipe, and the consequent formation of air bubbles, it was necessary to keep the stand-pipe full at all times; but, as the quantity taken out for experimental purposes varied, and was usually smaller than that taken in over the standard weir, the 12-in. regulating gate on the second floor was used. This gate discharged the water outside the building, as shown in Fig. 1, Plate XXIII. Alongside this gate a steam-gauge was connected to the stand-pipe. A few trials showed the pressure for which the stand-pipe became full.

In order to secure a steady head, the method of operation was, first, to adjust the 12-in regulating gate until the head indicated by the steam-gauge was nearly constant. Then the required head, as shown by the Pitot tube, was set by the 12-in. gate at the bottom of the experimental pipe. Finally, the level of the water in the stand-pipe was readjusted by the regulating gate, until the head was absolutely steady. In this way a uniform flow was assured

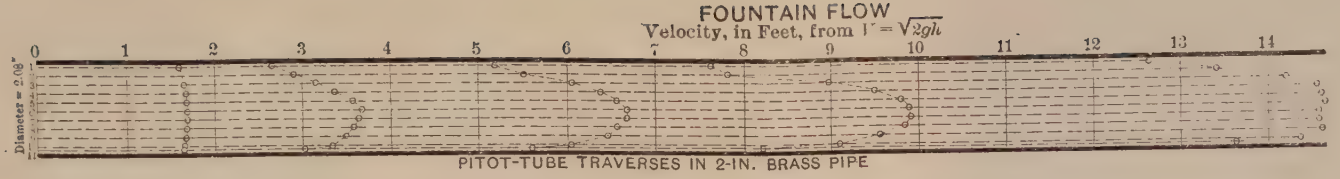
in the experimental pipe. At first, this adjustment took considerable time, but later it was made more quickly.

Method of Experimentation.—In making a run, the procedure was as follows: The Pitot tube was set with the impact opening directed vertically downward at the center of the pipe. The setting was done with a stick, on which was a mark placed so that when the end of the stick was held against the pipe under the tube sheath, and the edge of the indicator was brought over the mark, the impact opening of the tube was then on center. The direction of the impact hole of the tube was given by the pointer (already described). The air was then shaken out of the hose, and the head adjusted, as previously described.

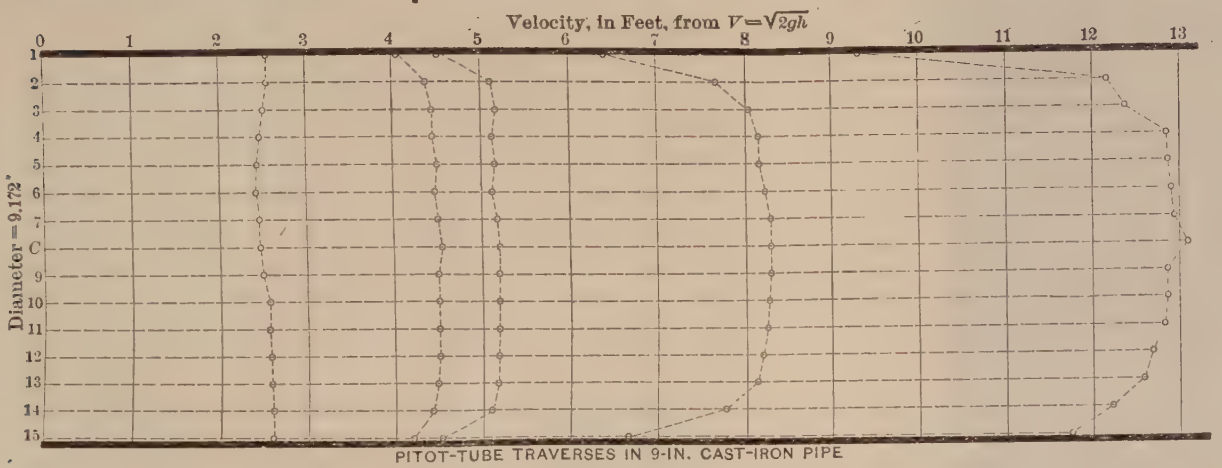
Two readings were made for each tape on the measuring tank, and recorded, unless the differences from the last level in the tank varied more than 0.004 ft., in which case the readings were repeated.

At a given signal, the diverter was swung out and the water allowed to flow into the tank. The initial time was noted by an ordinary well-regulated watch, and recorded. As soon as the run had started, readings on the glass-gauge were taken. The water column vibrated considerably at times, especially for high heads, when the jet would leap up and suddenly drop down, while at other times it remained quite stationary in an intermediate position. The maximum and minimum readings were taken, and especially the point where the jet remained stationary for a short time. During this latter stage a slight trembling was noticed in the gauge column. Usually, six readings were taken. These were distributed so that the final one was near the end of the run. All readings were made on the bottom of the meniscus.

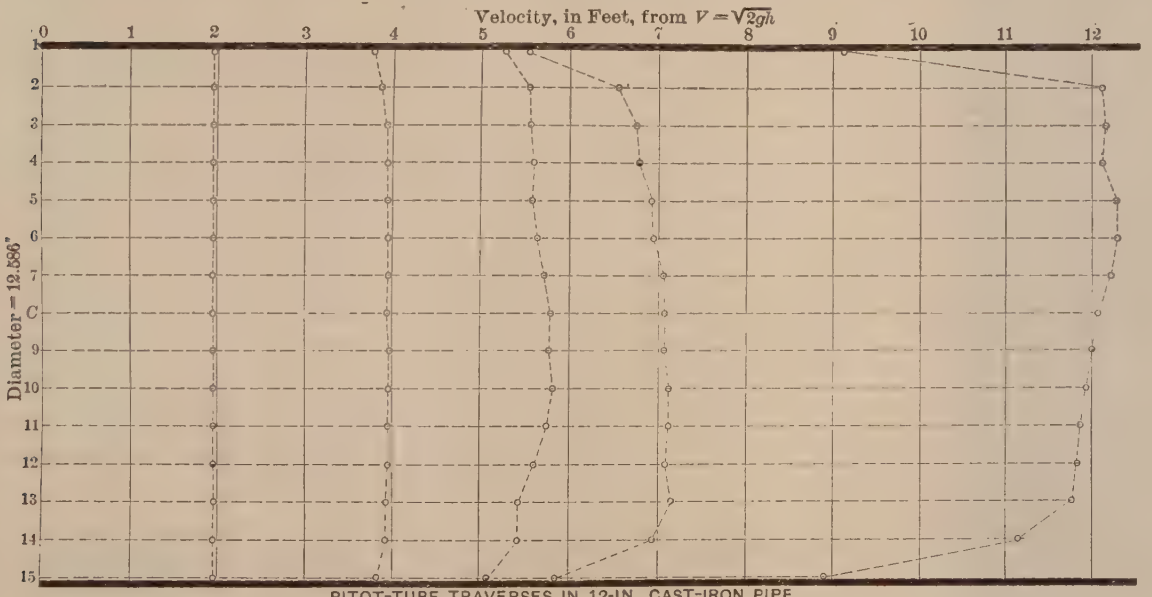
At the same time, observations were made on the sighting rods. This was done by holding the small blade of a knife at some point on the rod and standing off at arm's length. Then, by sighting along the knife to a point of equal reading on the other rod it could be determined whether the knife was too low or too high. The final level was chosen so that the fountain column would remain above it about the same length of time as below, or



PITOT-TUBE TRAVERSES IN 2-IN. BRASS PIPE



PITOT-TUBE TRAVERSES IN 9-IN. CAST-IRON PIPE



PITOT-TUBE TRAVERSES IN 12-IN. CAST-IRON PIPE

that a short, high leap would compensate for a smaller depression, but of longer duration.

One observation was made by each of the two observers for each head. The results were not announced until both had been made. Thus the observations were independent, which tended to eliminate the personal equation.

When the last gauge reading was made for Pitot tube on center, the tube was pulled out of the pipe 3 in. back into the sheath so as to indicate the wall pressure. The indication of the gauge for this position of the tube is evidently the same as for a simple piezometer in the side of the pipe. The Pitot tube here merely serves in place of a hose connection to transmit the pressure to the gauge, from the sheath, which corresponds to a piezometer tap. These readings, when corrected for the zero of the Pitot-tube gauge, give what are called "Pressure Heads" in Table 3. Two readings were made for each head. On the second signal, the diverter was pushed back into place, and the water was wasted over the side of the tank. The time was again noted.

The cutting edge of the diverter (Fig. 5) was made so that very little water was lost by splashing. Wooden guides along the side secured a sure return to the exact place.

While waiting for the water in the tank to quiet, another head was adjusted, which saved considerable time. The tapes were then read, as before, and another run started, unless the tank was full, in which case it was emptied by means of an 8-in. gate near the bottom. Enough water was left in to cover the bottom completely, and the tapes were then read again. The durations of the runs varied considerably. For low heads, runs of 25 min. were sometimes made, while for high heads, they were very short, especially with the 12-in. pipe, where, for the highest heads, the large tank was filled in 1 min. During the long runs one of the observers made frequent trips to the regulating gate to be sure that the head was steady, as indicated by the steam-gauge.

As the original notes fill one and one-half ordinary transit books, it was thought best to give herein only a sample form of notes. The original notes will be kept on record in the College of Civil Engineering, Cornell University.

SAMPLE FORM OF NOTES.

Runs for 4-in. pipe. Sept. 20th, 1905.

Zero of gauge = 0.231. Zero of rods = 3.17.

	Pitot-tube gauge.	Time.	Tapes.		Sighting rods.	Gauge reading for pressure.
			No. 1.	No. 3.		
Initial	1.580	11:34:45	6.940	26.959	4.48	0.296
	1.445		6.940	26.960		0.283
	1.530					
Final	1.550	11:37:15	7.709	27.730	4.45	0.285
	1.475		7.710	27.730		

Pitot-Tube Traverses.—Five Pitot-tube velocity traverses, Plates XXV and XXVI, at as many different heads, were made on each pipe, and distributed so as to show the character of the flow at the various heads. A stick was made (the same as for adjusting the Pitot tube on center) on which the diameter was laid off and divided into 15 equal parts, giving 16 points (except in the case of the 2 and 4-in. pipe, where 11 points were used). By placing the end of a stick against the pipe, under the tube sheath, any point along the diameter could be set from the edge of the pointer on the tube.

After adjusting the head as for the runs, the head on center was first read, three readings usually being taken. Then three readings were taken on the other points, from 1 to 16, and finally at the center again. The plottings (Plates XXV and XXVI) were made by using velocities from the formula, $V = \sqrt{2gh}$, and distances along the diameter.

As explained previously, the diaphragm was placed at the position on the elbow where the water assumed a vertical flow, namely, on the horizontal flange of the elbow, so that a more even distribution of velocities across the pipe would result. The long vertical length of pipe was used in order to give greater facility to the water in regaining its natural flow by the time it reached the crest of the pipe. It is apparent, therefore, that, to obtain the actual distribution of velocities, it was necessary to take points across the pipe, beginning at a point farthest away from the standpipe and running across to the nearest position (Fig. 5);

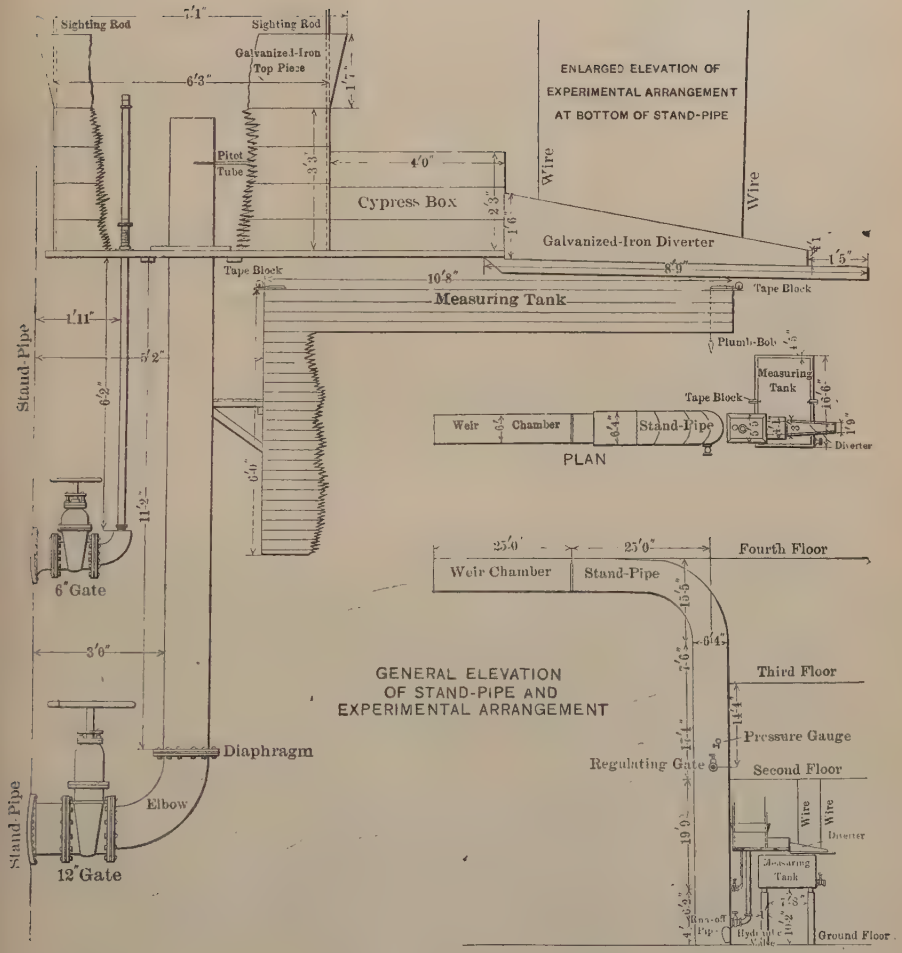

Sighting Rod
Sighting Rod
Galvanized-Iron Top Piece
Pitot Tube
Cypress Box
Tape Block
Measuring Tank
Stand-Pipe
6"Gate
3'0"
Diaphragm
Elbow
12"Gate
ENLARGED ELEVATION OF EXPERIMENTAL ARRANGEMENT AT BOTTOM OF STAND-PIPE
Wire
Galvanized-Iron Diverter
Plumb-Bob
Measuring Tank
Weir Chamber
Stand-Pipe
Diverter
PLAN
Fourth Floor
Third Floor
Second Floor
Ground Floor
Pressure Gauge
Regulating Gate
GENERAL ELEVATION OF STAND-PIPE AND EXPERIMENTAL ARRANGEMENT

FIG. 5.

in other words, in a line parallel to the flow of water from the stand-pipe to the vertical pipe, and hence parallel to the horizontal length of the elbow.

The traverse plottings (Plates XXV and XXVI) show a remarkably good distribution.

Rating Traverses.—In order to determine the tube coefficient, it was rated near the lower end of a 5-in. seamless drawn-brass pipe, 120 ft. long, leading directly from Beebe Lake and discharging into a concrete measuring tank of 500 cu. ft. capacity. These were made in co-operation with Mr. T. J. Rodhouse,* who was making an extensive series of tests on various Pitot tubes for the degree of M. C. E., and who, with Mr. C. Berry, was rating a great many tubes for the Ontario Power Company. These two men generously aided in the ratings, so that the writers feel that they are very reliable.

The cross-section of the pipe was divided into ten concentric rings of equal area, giving, with the center point, 21 points on a diameter. This method of dividing the area makes the calculation of mean velocity very easy by using the formula,

$$V_m = \left\{ \frac{V_1 + V_{21}}{2} + V_2 + V_3 + \ldots + V_{20} \right\}$$

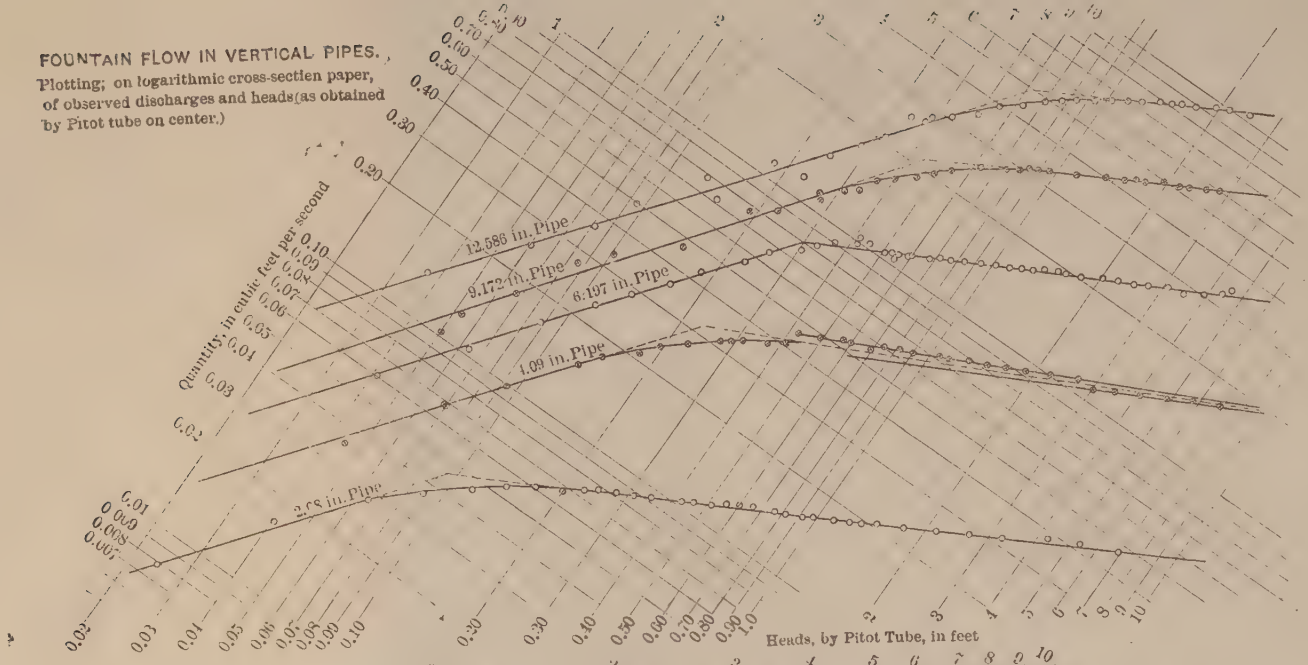
Wall pressures were taken from a tap in the same cross-section plane as the Pitot tube, and were connected to one arm of a water differential gauge, while the Pitot tube was connected to the other arm. The difference between the two column readings gave the velocity head, which was converted into velocity by the formula, $V = \sqrt{2\,g\,h}$.

While readings were being made on the gauges, several runs into the measuring tank were made for quantity. The average quantity, in cubic feet per second, during a traverse, divided by the area of the pipe section, gave the mean velocity, U (or actual), by tank measurement. The mean velocity, V_m, by the Pitot tube was computed from the foregoing formula, and, finally, the tube coefficient, $\frac{U}{V_m}$, was obtained. This was found to be 1.005 for the mean of five traverses, as shown in Table 2, This is very near unity, and, for all practical purposes, may be taken as such.

* See Thesis by Mr. T. J. Rodhouse, Cornell College of Civil Engineering, 1905.

FOUNTAIN FLOW IN VERTICAL PIPES.

Plotting; on logarithmic cross-section paper, of observed discharges and heads (as obtained by Pitot tube on center.)



FOUNTAIN FLOW IN VERTICAL PIPES.

Plotting, on logarithmic cross-section paper, of observed discharges and heads (as obtained by sighting rods.)

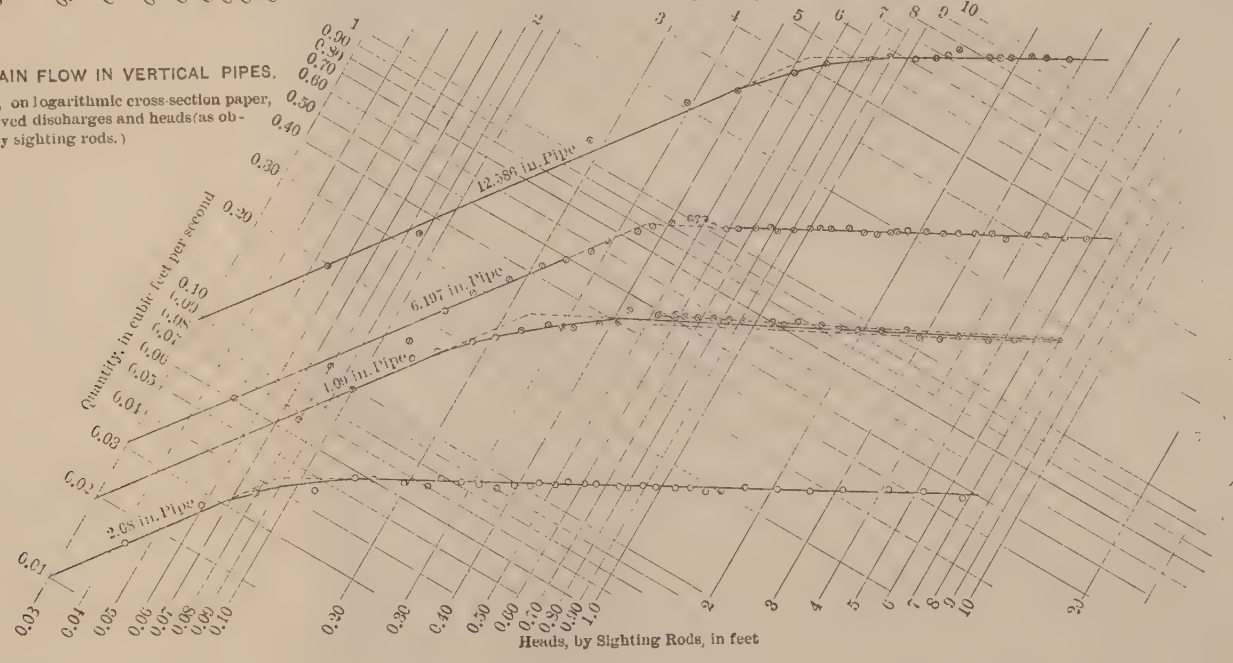


TABLE 2.—PITOT-TUBE RATING IN 5-IN. BRASS PIPE LINE.

Number of Traverse.	U	V_m	$\frac{U}{V_m}$	V_c	$\frac{V_m}{V_c}$	$\frac{U}{V_c}$
1	5.244	5.298	0.992	6.110	0.865	0.858
2	4.125	3.956	1.043	4.770	0.830	0.865
3	7.760	7.800	0.995	8.930	0.873	0.870
4	3.495	3.530	0.990	4.200	0.841	0.833
5	1.500	1.486	1.009	1.810	0.821	0.829
Averages	.....	.....	1.005	.....	0.846	0.851

TABLE 2.—*Continued.*

ADDITIONAL RUNS WITH PITOT TUBE KEPT ON CENTER.

Number of Run.	V_c	U	$\frac{U}{V_c}$
1	10.43	9.13	0.875
2	9.45	8.24	0.872
3	7.84	6.83	0.872
4	6.62	5.67	0.858
5	4.53	3.90	0.862
Average	.....	.....	0.868

V_c = Velocity of tube at center,
V_m = Mean velocity from traverses,
U = Mean velocity (measured).

The writers believe that for most work on pipes in actual practice, a simple tube of this form gives results fully as accurate as, if not better than, a great many of the more complex but less durable tubes. The ratio, $\frac{V_m}{V_c}$, averages 0.846, which agrees closely with the results by other observers.*

The writers do not present in detail the numerical data for either set of traverses, as the plottings are sufficiently illustrative. The data, however, will be kept on record in the College of Civil Engineering of Cornell University.

* Paper by Messrs. Williams, Hubbell and Fenkel, with discussion; *Transactions*, Am. Soc. C. E., Vol. XLVII.

Reduction of Observations and Results.

Computations.—The averages of the heads, read by the Pitot tube on center and the sighting rods, were first corrected for the zero readings. Then, by the tank rating curve, the quantity for each run was picked off for the corresponding tape readings. The average of the quantities obtained by the two tapes divided by the length of the run in seconds gave the quantity, Q, in cubic feet per second. The results of these computations are shown in Table 3, in which the pressures (corrected for zero reading) are also shown. These were not taken for all heads, because, when the gauge was raised, the water column was below the gauge. At other times, when a series of runs was rejected (which was done on several occasions in the field because of air in the tube), the pressure observations were not repeated with the other measurements. The writers plotted these pressure heads with respect to the Pitot-tube heads, but could find no regular relation. The irregularity of these pressure heads shows how unreliable they are as a means of measurement. Had they shown any regularity, they would have been used to subtract from the Pitot-tube head, in order to obtain the velocity heads. For this reason the tube could not be rated on the experimental pipes, and the traverse plottings (Plates XXV and XXVI) serve only to show the distribution of velocities. The rating on the 5-in. brass pipe is sufficient to show the action of the two.

Plottings.—The values of H and Q were then plotted (Plate XXVII) with the values of H as abscissas and the values of Q as ordinates. The average line through a series of points gives an equation of the form, $Q = m\ H^n$. Table 4 gives the values of m and n for the weir and jet conditions of each pipe.

The average lines were selected as follows: It was noted by inspection that the upper and lower parts of the series of points fell generally along separate straight lines with different slopes. Then, by starting at the higher end, a line was drawn which balanced up the points on either side; and, in the same way, a line was drawn for the lower part of the series. Inside the vertex of the angle formed by these two lines, the points seemed to fall along a well-defined curve. After several trials, curves were drawn which appeared to be the same for all pipes.

TABLE 3.—Observed Quantities.

Experimental Runs on 2-in. Pipe. Zero of Pitot Tube = 0.188. Zero of Rods = 3.14.

Number of run.	Head by Pitot tube on center, in feet.	Head by sighting rods, in feet.	Pressure head, in feet.	Quantity discharged, in cubic feet per second.	Remarks.
1	0.024	0.020	0.022	0.0079	Used 5-cu. ft. tank and weighing scales for measurement of quantities in first five runs.
2	0.040	0.043	0.032	0.0163	
3	0.066	0.060	0.034	0.0279	
4	0.088	0.080	0.032	0.0362	
5	0.115	0.115	0.026	0.0456	Sheet form.
6	0.140	0.140	0.022	0.0538	Slightly disturbed.
7	0.166	0.160	0.020	0.0603	
8	0.198	0.195	0.020	0.0661	Rough surface, with oscillations of water column.
9	0.225	0.230	0.018	0.0731	
10	0.243	0.245	0.019	0.0781	
11	0.276	0.280	0.019	0.0828	
12	0.309	0.315	0.021	0.0869	
13	0.346	0.355	0.022	0.0927	
14	0.384	0.395	0.022	0.0988	
15	0.426	0.438	0.023	0.104	
16	0.453	0.455	0.023	0.109	
17	0.512	0.510	0.024	0.115	
18	0.558	0.550	0.028	0.124	
19	0.597	0.610	0.029	0.129	
20	0.655	0.660	0.031	0.136	
21	0.760	0.770	0.033	0.145	
22	0.816	0.815	0.033	0.148	
23	0.919	0.895	0.037	0.158	
24	0.979	0.975	0.042	0.164	
25	1.114	1.095	0.048	0.175	
26	1.234	1.205	0.054	0.185	
27	1.333	1.345	0.056	0.192	
28	1.477	1.460	0.057	0.202	
29	1.749	1.695	.....	0.223	Gauge raised 1 ft.
30	2.140	2.035	.....	0.251	
31	2.660	2.535	.....	0.281	Fountain jet condition apparent to the eye.
32	3.245	3.110	.....	0.317	
33	4.270	4.135	.....	0.379	
34	5.335	5.135	.....	0.419	
35	6.927	6.760	.....	0.470	

Experimental Runs on 4-in. Pipe. Zero of Pitot Tube = 0.231. Zero of Rods = 3.17.

Number of run.	Head by Pitot tube on center in feet.	Head by sighting rods, in feet.	Pressure head, in feet.	Quantity discharged, in cubic feet per second.	Remarks.
1	0.044	.....	0.040	0.035	Bulb form of discharge.
2	0.068	.....	0.058	0.067	Sheet form.
3	0.070	0.080	0.058	0.067	
4	0.093	0.105	0.068	0.100	
5	0.130	0.130	0.075	0.151	
6	0.145	0.150	0.072	0.174	
7	0.176	0.180	0.071	0.209	Water column unsteady.
8	0.195	0.205	0.066	0.234	
9	0.228	0.235	0.060	0.270	
10	0.274	0.270	0.055	0.310	
11	0.291	0.300	0.052	0.323	
12	0.312	0.320	0.052	0.342	
13	0.367	0.375	0.044	0.375	
14	0.407	0.415	0.043	0.407	
15	0.419	0.425	0.060	0.453	Assuming fountain jet form.
16	0.491	0.520	0.051	0.488	
17	0.563	0.590	0.051	0.523	
18	0.597	0.620	0.049	0.533	Jet form established.
19	0.681	0.680	0.047	0.555	
20	0.728	0.755	0.047	0.584	
21	0.813	0.825	0.047	0.619	
22	0.888	0.905	0.050	0.644	
23	1.061	1.080	0.153	0.712	
24	1.138	1.155	0.155	0.728	Unsteady at about 1.3.
25	1.285	1.295	0.057	0.783	
26	1.465	1.500	0.058	0.880	Gauge raised.
27	1.648	1.700	.....	0.872	Great fluctuation noted on very slight motion of gate.
28	1.888	1.930	.....	0.936	
29	2.218	2.230	.....	1.008	
30	2.658	2.630	.....	1.101	
31	3.021	2.905	.....	1.103	
32	3.482	3.330	.....	1.178	
33	4.072	3.780	.....	1.276	
34	4.824	4.530	.....	1.393	
35	5.681	5.355	.....	1.540	
36	6.845	6.230	.....	1.655	

TABLE 3.—(*Continued*).

EXPERIMENTAL RUNS ON 9-IN. PIPE. ZERO OF PITOT TUBE = 0.052.

NO RODS OR PRESSURES TAKEN ON THIS PIPE. PRESSURE GAUGE AT 11.4 LB. (STAND-PIPE).

Number of run.	Head by Pitot tube on center, in feet.	Head by sighting rods, in feet.	Pressure head, in feet.	Quantity discharged, in cubic feet per second.	Remarks.	Number of run.	Head by Pitot tube on center, in feet.	Head by sighting rods, in feet.	Pressure head, in feet.	Quantity discharged, in cubic feet per second.	Remarks.
1	0.049	.....	.....	0.104	Sheet discharge.	19	0.535	.....	.....	2.206	
2	0.052	.....	.....	0.125	Smooth discharge.	20	0.626	.....	.....	2.542	
3	0.068	.....	.....	0.178	Slightly rough.	21	0.656	.....	.....	2.508	
4	0.087	.....	.....	0.274		22	0.734	.....	.....	2.779	
5	0.102	.....	.....	0.334		23	0.799	.....	.....	2.926	
6	0.147	.....	.....	0.470	Rough.	24	0.851	.....	.....	3.070	
7	0.148	.....	.....	0.717		25	0.897	.....	.....	3.155	
8	0.188	.....	.....	0.771		26	0.942	.....	.....	3.246	
9	0.222	.....	.....	0.832		27	1.141	.....	.....	3.596	
10	0.223	.....	.....	0.858		28	1.357	.....	.....	3.987	
11	0.269	.....	.....	1.130		29	1.532	.....	.....	4.246	
12	0.278	.....	.....	1.094	Velocity at edge greater	30	1.623	.....	.....	4.395	
13	0.307	.....	.....	1.274	than at center on one	31	1.749	.....	.....	4.543	
14	0.331	.....	.....	1.362	side of pipe.	32	1.947	.....	.....	4.853	
15	0.358	.....	.....	1.536		33	2.172	.....	.....	5.113	
16	0.391	.....	.....	1.658		34	2.353	.....	.....	5.358	
17	0.443	.....	.....	1.833		35	2.564	.....	.....	5.593	
18	0.493	.....	.....	2.052		36	2.768	.....	.....	5.841	

TABLE 3.—(*Continued*).

EXPERIMENTAL RUNS ON 6-IN. PIPE. ZERO OF PITOT TUBE = 0.144. ZERO OF RODS = 3.07.

Number of run.	Head by Pitot tube on center, in feet.	Head by sighting rods, in feet.	Pressure head, in feet.	Quantity discharged, in cubic feet per second.	Remarks.
1	0.041	0.050	.....	0.0607	Barely flowing over edge.
2	0.063	0.080	.....	0.1049	Sheet form.
3	0.088	0.120	.....	0.167	
4	0.112	0.135	.....	0.230	Slight disturbance.
5	0.132	0.150	.....	0.287	
6	0.160	0.180	.....	0.355	
7	0.184	0.210	.....	0.432	
8	0.205	0.240	.....	0.495	Violent disturbance, swirls in circle, great variation of head.
9	0.228	0.270	.....	0.565	
10	0.253	0.290	.....	0.650	
11	0.306	0.340	.....	0.756	Occasional spouts.
12	0.330	0.370	.....	0.832	
13	0.360	0.410	.....	0.917	
14	0.410	0.450	.....	1.002	
15	0.412	0.470	.....	1.029	
16	0.446	0.470	.....	1.030	
17	0.447	0.490	.....	1.048	
18	0.507	0.530	.....	1.039	
19	0.537	0.590	.....	1.077	
20	0.556	0.580	.....	1.098	
21	0.590	0.630	.....	1.127	
22	0.600	0.630	.....	1.139	Column jumps as much as 0.1 ft. Would seem to indicate a critical velocity.
23	0.671	0.710	.....	1.203	
24	0.715	0.770	.....	1.255	
25	0.760	0.830	.....	1.293	
26	0.840	0.900	.....	1.364	Not so much spreading noticed as in 9- and 12-inch pipes.
27	0.931	1.000	.....	1.452	
28	1.015	1.070	.....	1.518	
29	1.105	1.150	.....	1.576	
30	1.216	1.290	.....	1.675	
31	1.301	1.430	.....	1.736	Head fluctuating considerably.
32	1.385	1.550	.....	1.811	
33	1.508	1.680	.....	1.920	
34	1.595	1.740	.....	1.949	Gauge raised.
35	1.760	1.870	.....	2.032	
36	2.030	2.100	.....	2.201	
37	2.337	2.304	.....	2.309	
38	2.501	2.590	.....	2.420	
39	2.756	2.880	.....	2.558	
40	3.098	3.180	.....	2.713	
41	3.462	3.550	.....	2.784	
42	3.908	4.000	.....	3.035	
43	4.357	4.430	.....	3.223	
44	4.557	5.030	.....	3.437	
45	5.746	5.800	.....	3.704	
The following series (Pitot Tube on center, and Pressure readings) taken to show how pressure acts.					
.....	0.034	.....	0.027	.....	
.....	0.066	.....	0.056	.....	
.....	0.097	.....	0.076	.....	
.....	0.119	.....	0.092	.....	
.....	0.142	.....	0.084	.....	
.....	0.173	.....	0.102	.....	
.....	0.201	.....	0.100	.....	
.....	0.225	.....	0.121	.....	
.....	0.256	.....	0.125	.....	
.....	0.288	.....	0.102	.....	
.....	0.365	.....	0.141	.....	
.....	0.402	.....	0.141	.....	
.....	0.457	.....	0.141	.....	

TABLE 3.—(*Continued*).

EXPERIMENTAL RUNS ON 12-IN. PIPE. ZERO OF PITOT TUBE = 0.069. ZERO OF RODS = 3.04.

Number of run.	Head by Pitot tube on center, in feet.	Head by sighting rods, in feet.	Pressure head, in feet.	Quantity discharged in cubic feet per second.	Remarks.
1..	0.036	.....	.....	0.139	At 1 ft. a whirling motion observed. At times, when near one edge, the water would shoot across.
2..	0.046	0.055	.....	0.197	
3..	0.060	.....	.....	0.250	
4..	0.081	0.087	0.123	0.396	
5..	0.095	.....	.....	0.499	
6..	0.129	.....	.....	0.787	Sheet form.
7..	0.180	0.180	0.205	1.127	Slightly rough.
8..	0.213	.....	.....	1.192	Between 0.2 and 0.3 the flow would not settle.
9..	0.245	.....	.....	1.477	
10..	0.284	.....	.....	1.760	Below 0.235 trial by Pitot tube indicated that velocity at center was less than at edge; above this the center velocity was higher; very unsteady; appears to be a critical velocity.
11..	0.317	0.290	0.227	2.051	
12..	0.336	.....	.....	2.602	
13..	0.376	0.380	0.235	2.673	
14..	0.386	.....	.....	2.804	
15..	0.431	.....	.....	3.070	
16..	0.500	.....	.....	3.487	
17..	0.552	.....	.....	3.922	Getting unsteady.
18..	0.627	.....	.....	4.309	
19..	0.705	.....	.....	4.868	Bubbling begins.
20..	0.770	.....	.....	5.213	
21..	0.832	.....	.....	5.552	
22..	0.928	.....	.....	5.960	
23..	1.030	.....	.....	6.367	
24..	1.130	.....	.....	6.759	
25..	1.232	.....	.....	7.128	
26..	1.378	1.340	0.107	7.698	
27..	1.481	1.650	0.111	8.073	
28..	1.560	1.760	0.101	8.358	
29..	1.729	1.890	0.089	8.754	
30..	1.953	2.150	.....	9.551	Gauge raised.
31..	2.131	2.360	.....	9.866	
32..	2.441	2.790	.....	10.709	
	The following series was run to show the action of the pressure head.				
....	0.521	0.510	0.226		
....	0.631	0.610	0.206		
....	0.774	0.780	0.176		
....	0.873	0.890	0.153		
....	0.975	1.030	0.139		
....	1.099	1.180	0.116		
....	1.219	1.260	0.106		

ne the relation between the
. The values of m are
s (see Plate XXVII).

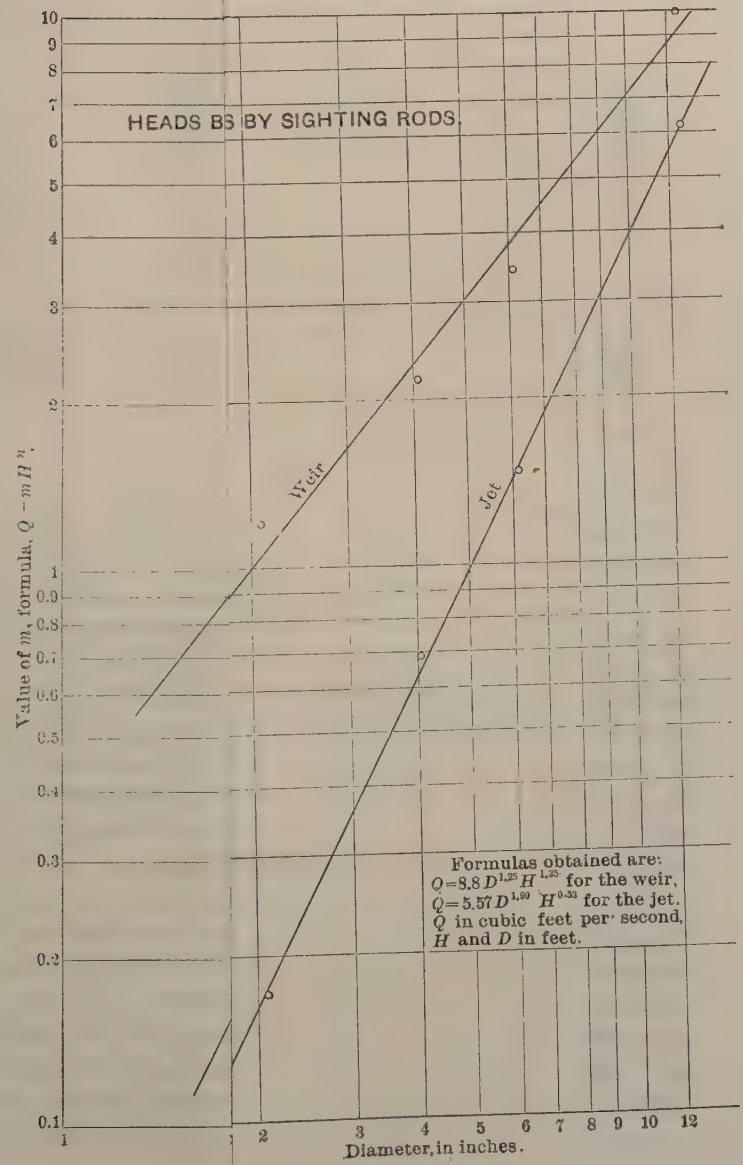

HEADS BS BY SIGHTING RODS.
Weir
Jet
Value of m, formula, $Q = m H^n$.
Diameter, in inches.
Formulas obtained are:
$Q = 8.8\, D^{1.25} H^{1.35}$ for the weir,
$Q = 5.57\, D^{1.99} H^{0.33}$ for the jet.
Q in cubic feet per second,
H and D in feet.

TABLE 4.—VALUES OF m AND n IN THE FORMULA, $Q = m H^n$.

For heads read with Pitot tube on center.

Diameter of pipe, in inches.	Weir.		Jet.	
	m	n	m	n
2.08	1.00	1.29	0.168	0.54
4.09	2.08	1.28	0.660	0.496
6.197	3.85	1.29	1.495	0.52
9.172	6.45	1.31	3.380	0.548
12.586	9.40	1.27	6.500	0.54
Averages	.....	1.29	.....	0.53

For heads read with sighting rods.

2.08	1.20	1.352	0.168	0.55
4.00	2.34	1.351	0.680	0.50
6.197	3.40	1.337	1.470	0.53
12.586	10.60	1.341	6.130	0.56
Averages	.....	1.345	.....	0.53

As was expected, the transition from the weir to the jet condition does not take place suddenly, but very gradually, as shown by the curved lines. This result agrees with the appearance of the actual flow, where, when the head is steadily increased, it takes considerable time to pass from the weir condition to the actual jet.

The only case which is an exception is in the 6-in. pipe. In the plotting of this pipe, there is seen an apparently sudden change from the weir to the jet. There is a sudden bend where an increase in head does not give a corresponding increase in the quantity. It is hardly reasonable to suppose that this same condition would hold for all pipes of the same diameter, yet an explanation is difficult. When the writers noted it on the plottings they made several runs on either side and at the bend, but the result was the same as before, so that experimental error could not be held responsible. The only explanation which the writers can offer for the phenomenon is this: The inside of the pipe was machine-worked somewhat more than the other pipes, and, instead of coming gradually in to the former diameter, there was a slightly rounded angle. The diameter near the top was 0.1 in. larger than that 6 in. down. Thus, when the flow became a jet the column may have shot past the wall,

unsupported, as it does outside the pipe, so that the real diameter was the small one farther down. If this be so, the quantity, in cubic feet per second, should be increased. In other words, the run would actually represent that from a pipe of smaller diameter.

The jet of the 4-in. pipe shows two well-defined lines, which may have been due to the cause mentioned above. The line representing the average of these two lines was the one finally adopted.

The plottings of quantities and heads read by sighting rods (Plate XXVIII) represent similar conditions, except that the values of m and n are higher for the weir, and, in most cases, lower for the jet. If the Pitot tube is a true indicator, the rods were read too low for the low head and too high for the high head. The sudden jumping of the water creates an impression on the eye that the average head is somewhat higher than is actually the case. For very low heads the reflection of light on the water surface has considerable effect on the rod readings. The writers would advise the use of the hook-gauge or other means for these low heads when a Pitot tube is not available. The latter enables one to make accurate readings for these low heads, as there is very little vibration.

The points where the intermediate conditions begin and leave off are plotted (Fig. 6), for each diameter, on logarithmic cross-section paper, with the heads as ordinates and the diameters as abscissas; also, the points of intersection for the jet and weir lines were plotted for each pipe. The equations of the lines through these points can be used for determining the heads at which the intermediate conditions begin and end, for any size of pipe. The equations are:

$H = 0.028\ D^{1.04}$, for the end of the weir condition;

$H = 0.107\ D^{1.03}$, for the beginning of the jet condition;

$H = 0\ 045\ D^{1.04}$, for the intersection of weir and jet lines.

Here, H is in feet, and D in inches.

The fact that these lines are so nearly parallel and have almost unity slope shows that the period of intermediate condition is relatively the same for all pipes. The flows from the smaller pipes go through the transition quicker than those from the larger ones. The actual differences in heads between the beginning and end vary almost directly as the diameter.

Final Plottings.—The values of m, in the formula, $Q = m\ H^n$,

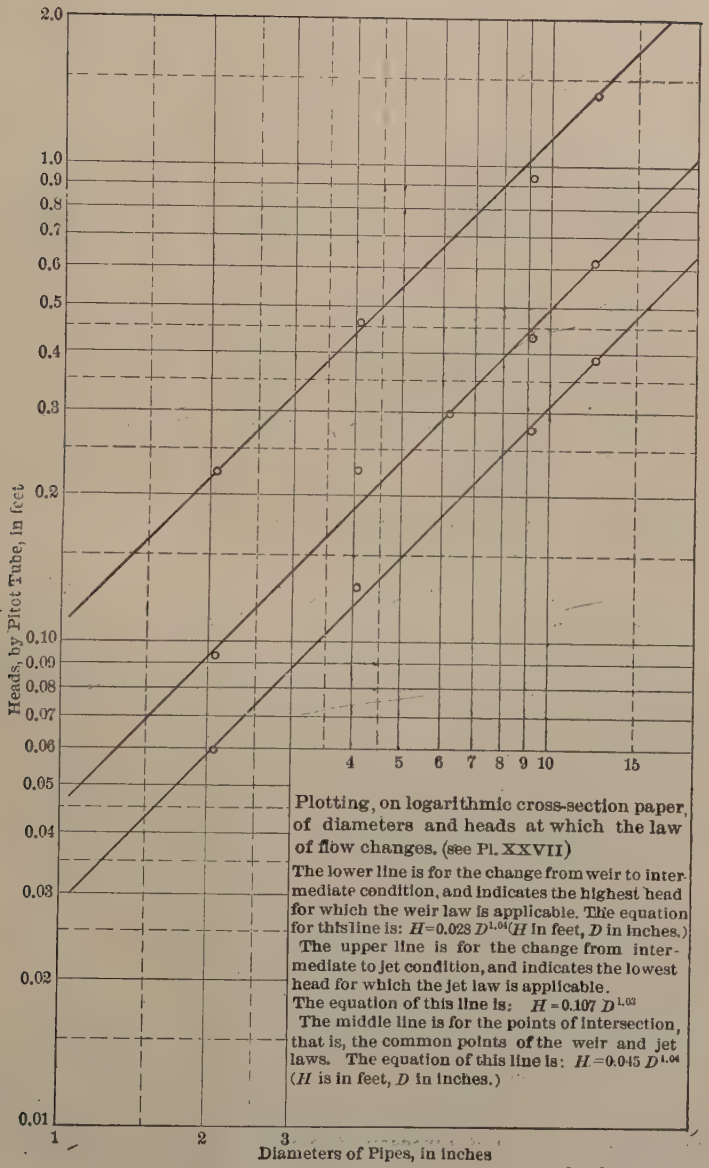

2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
Heads, by Pitot Tube, in feet
1
2
3
4
5
6
7
8
9
10
15
Diameters of Pipes, in inches
Plotting, on logarithmic cross-section paper, of diameters and heads at which the law of flow changes. (see Pl. XXVII)
The lower line is for the change from weir to intermediate condition, and indicates the highest head for which the weir law is applicable. The equation for this line is: $H=0.028\,D^{1.04}$ (H in feet, D in inches.)
The upper line is for the change from intermediate to jet condition, and indicates the lowest head for which the jet law is applicable. The equation of this line is: $H=0.107\,D^{1.03}$
The middle line is for the points of intersection, that is, the common points of the weir and jet laws. The equation of this line is: $H=0.045\,D^{1.04}$ (H is in feet, D in inches.)

FIG. 6.

from Table 4, were then plotted for each diameter (Plate XXVIII) on logarithmic cross-section paper for the Pitot tube and sighting rods. The values of m were plotted as ordinates and the values of D as abscissas. Lines through these points gave equations of the form, $m = x\ D^k$, which, substituted in $Q = m\ H^n$, gave as the final form, $Q = x\ D^k\ H^n$, in which n is the average value taken from Table 4.

It was not attempted to fit any formula to the intermediate condition. Tables, which are of more practical value, were made out.

Adjustment of Formulas.—Using the general equation, $Q = x\ D^k\ H^n$, the heads as observed in the individual experiments were substituted, together with the corresponding diameters. This computed quantity was compared with the observed quantity, and coefficients, $\frac{Q \text{ (by formula)}}{Q \text{ (observed)}}$, were obtained. The averages of these coefficients for the weir and jet conditions of each pipe were then computed. From these average coefficients it was possible to adjust the formula so as to give an equation with the least mean error. All the points were considered except where there was clinging discharge, caused by a vacuum beneath the sheet of water. This clinging discharge generally gives a greater discharge for the same head.

The formulas finally adopted are:

For heads read with Pitot tube,

$$Q = 8.8\ D^{1.29}\ H^{1.29}, \text{ for weir flow} \quad (1)$$

$$Q = 5.84\ D^{2.025}\ H^{0.53}, \text{ for the jet flow} \quad (2)$$

For heads read with sighting rods,

$$Q = 8.8\ D^{1.25}\ H^{1.35}, \text{ for weir flow} \quad (3)$$

$$Q = 5.57\ D^{1.99}\ H^{0.53}, \text{ for the jet flow} \quad (4)$$

Q is in cubic feet per second; H and D are in feet.

A graphical representation of the differences between the individual experiments and the general formulas is given in Figs. 7, 8 and 9. The maximum variations indicated in this plotting and in Table 5 do not represent the probable error to be expected in using the general formula. The average departures for each pipe are given in Table 5.

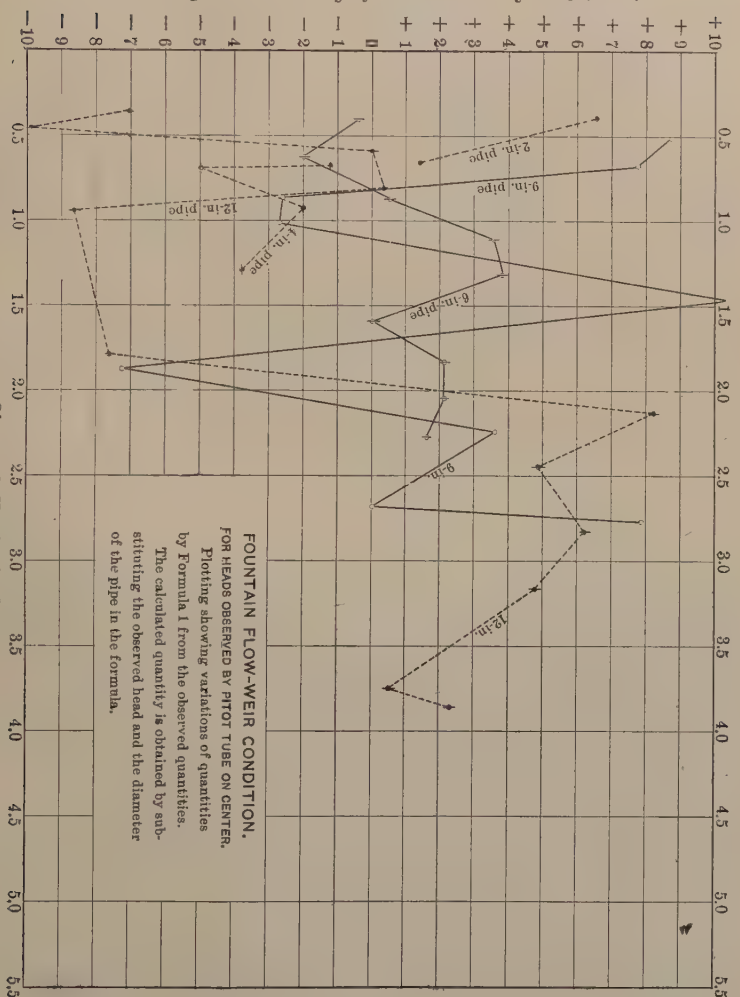

Percentage Variation of Q by Formula from Q as observed.
+10
+9
+8
+7
+6
+5
+4
+3
+2
+1
0
−1
−2
−3
−4
−5
−6
−7
−8
−9
−10
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
2-in. pipe
9-in. pipe
12-in. pipe
4-in. pipe
6-in. pipe
9-in.
12-in.
FOUNTAIN FLOW-WEIR CONDITION.
FOR HEADS OBSERVED BY PITOT TUBE ON CENTER.
Plotting showing variations of quantities by Formula 1 from the observed quantities.
The calculated quantity is obtained by substituting the observed head and the diameter of the pipe in the formula.
Observed Heads, in feet.

Fig. 7.

TABLE 5.—Comparison of Results, by General Formula, with Experimental Results.

For heads read with the Pitot tube on center.

Diameter of pipe, in inches.	Average Values of $\frac{Q \text{ (Computed)}}{Q \text{ (Observed)}}$		Maximum Departure of Formula from Individual Experiment.	
	Weir.	Jet.	Weir.	Jet.
2.08	0.961	1.010	−6.6%	−3.9%
4.09	1.031	1.008	+5.0	−9.5
6.197	0.986	1.023	−3.9	−4.7
9.172	1.029	1.001	+8.8	−0.9
12.586	0.995	0.975	−9.9	+4.0
Means	1.000	1.003	.....	.....

For heads read with sighting rods.

2.08	0.915	1.020	−9.3%	−5.8%
4.09	1.046	0.999	+11.8	−5.1
6.197	1.071	1.027	+16.8	−4.6
12.586	0.900	0.972	+18.0	+6.3
Means	0.983	1.004	.....	.....

Comparison of Results.—In comparing the formulas obtained with the table prepared by Professor Todd (mentioned previously) considerable variation was noticed. The table, which was based on the theoretical jet formula, is very inaccurate for low heads and slightly in error for the higher heads. The lower the head for any pipe, the greater the error. In a particular case, the table gives results for a head of 0.5 in. on a 12-in. pipe, 900% of the value obtained from the weir formula. The error decreases as the head increases; but even on the high heads the theoretical jet formula gives results which are too high. The theoretical formula is $Q = F\,V$, in which $F = \frac{\pi\,d^2}{4}$ = the area of the pipe, and V = velocity, from $V = \sqrt{2\,g\,h}$. These values, substituted, give $Q = \frac{\pi\,d^2}{4}\sqrt{g\,h}$, or $Q = 6.30\,d^2\,H^{0.5}$; while the jet formula, obtained from these experiments, is $Q = 5.84\,D^{2.025}\,H^{0.53}$.

Messrs. Elliot and Porter (mentioned previously) obtained for the weir condition, $Q = 2.11\,H^{1.327}$, for a pipe 4.174 in. in diameter. This

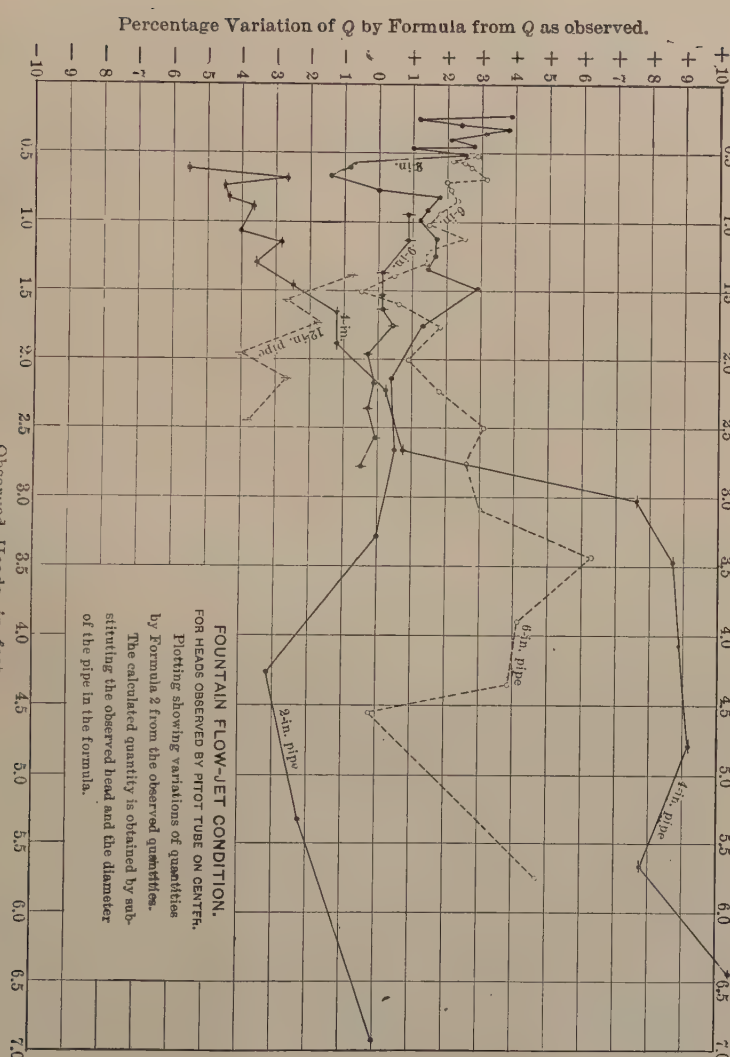

Percentage Variation of Q by Formula from Q as observed.
+10
+9
+8
+7
+6
+5
+4
+3
+2
+1
0
−1
−2
−3
−4
−5
−6
−7
−8
−9
−10
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
8 in.
6 in.
4 in.
12 in. pipe
6 in. pipe
2 in. pipe
4 in. pipe
FOUNTAIN FLOW-JET CONDITION.
FOR HEADS OBSERVED BY PITOT TUBE ON CENTER.
Plotting showing variations of quantities by Formula 2 from the observed quantities.
The calculated quantity is obtained by substituting the observed head and the diameter of the pipe in the formula.
Observed Heads, in feet.

FIG. 8.

diameter, substituted in the general weir formula ($Q = 8.8\ D^{1.29}\ H^{1.29}$), gives $Q = 2.25\ H^{1.29}$. This gives a fair check on the writers' general formula.

Diagram and Table for Practical Use of Results.—Plate XXIX is a diagram for obtaining the quantities, in cubic feet per second, for heads from 0.02 in. to 10 ft., and for pipes from 1 to 24 in. in diameter. It is made on logarithmic cross-section paper, and is based on the general jet and weir formulas. The lines for the intermediate condition are drawn exactly like the curves obtained in the experimental plottings, and the points at which it begins and ends are obtained from the equations given.

Table 6 gives numerical values taken from this diagram. The zigzag lines separate the intermediate-condition values from the weir and jet values.

Plate XXIX and Table 6 are based on measurements of heads with the Pitot tube on center. For sighting rods, a similar diagram and table can be readily made by using Formulas 3 and 4.

Conclusion.—In presenting this paper, the writers feel that considerable light is being thrown on a hitherto unexplored subject. While the jet formulas are well defined, there is still room for experimentation in the weir condition.

It has been pointed out that the hitherto existing knowledge is very limited, and often leads to inaccuracy.

The heads obtained by sighting rods, while giving large errors for low heads, give very good results for high heads. As compared with the Pitot tube results, the heads are slightly higher.

The Pitot tube gives very good results, especially for low heads, and can also be used to advantage in the field. It is a device which does not require very special knowledge for good results, and is easily constructed.

The Fountain Jet as a Measuring Device.—The writers believe that a fountain jet can be used as a measuring device, and will come well within the limits of accuracy of most hydraulic measurements. When used for this purpose, the diameter should preferably be of such size as to form a jet, in which case very accurate results may be obtained.

In general, measuring apparatus must be adapted to the expected conditions. Thus, if a nozzle were made too large, and if

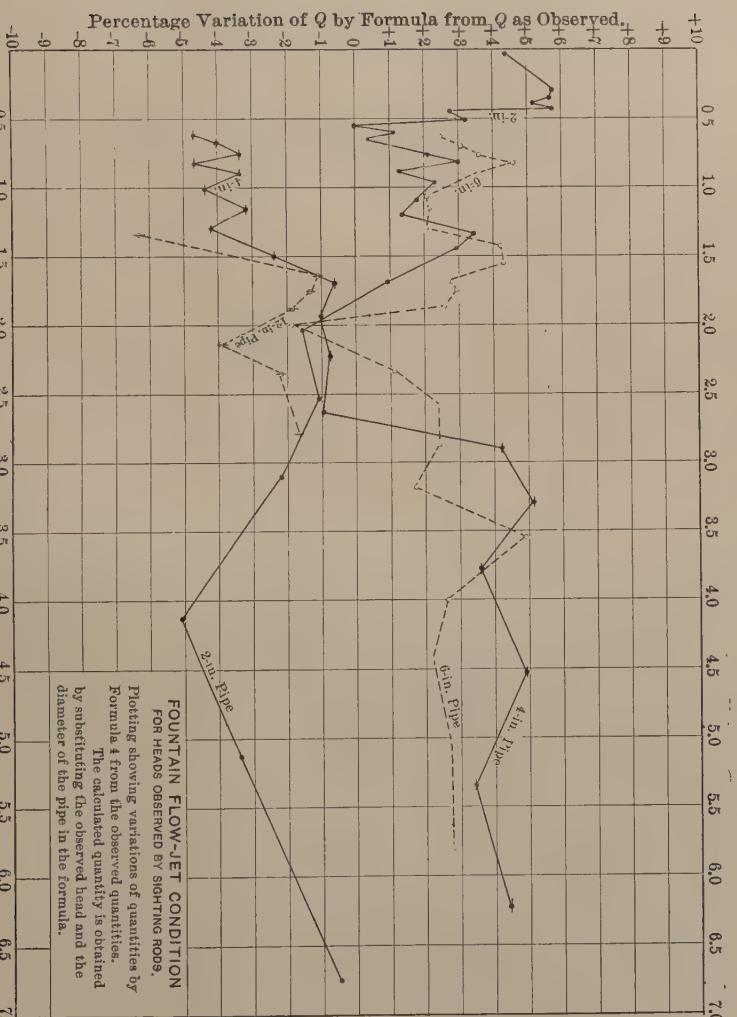

Percentage Variation of Q by Formula from Q as Observed.
+10
+9
+8
+7
+6
+5
+4
+3
+2
+1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
Observed Heads, in feet.
FOUNTAIN FLOW-JET CONDITION
FOR HEADS OBSERVED BY SIGHTING RODS.
Plotting showing variations of quantities by Formula 4 from the observed quantities.
The calculated quantity is obtained by substituting the observed head and the diameter of the pipe in the formula.
2-in. Pipe
4-in. Pipe
6-in. Pipe
12-in. Pipe
1-in.
0·5-in.
2-in.

Fig. 9.

the flow through it moved too slowly, the errors involved in using it for a measuring device might be considerable; for a certain head over a weir, there must be a proper height and length of weir for the best results; a Venturi meter must have its ratio of contraction depending on the range of velocities in the pipe line.

A fountain jet with sighting rods, or with a Pitot tube, is an exceedingly simple and cheap measuring device for the discharging end of a pipe line. Any form of Pitot tube having a coefficient of unity may be used. The form used by the writers is very easily constructed, and answers the purpose admirably. Adequate sighting rods are available almost anywhere.

Accuracy.—In using the formulas given, or the diagram, (Plate XXIX), or Table 6, it may be remarked that the writers' results (see Table 5 and Figs. 7, 8 and 9) indicate that the average accuracy to be expected for any particular pipe in measurements of fountain flow with the jet condition, using either sighting rods or the Pitot tube, is within 3% of the truth. For the weir condition, the average accuracy with the Pitot tube is within 4 per cent. The sighting rods give a maximum average error of 10% for the weir flow. A single measurement of fountain flow by either of these means cannot be considered reliable within twice the above stated maximum average errors.

PLATE XXIX.
TRANS. AM. SOC. CIV. ENGRS.
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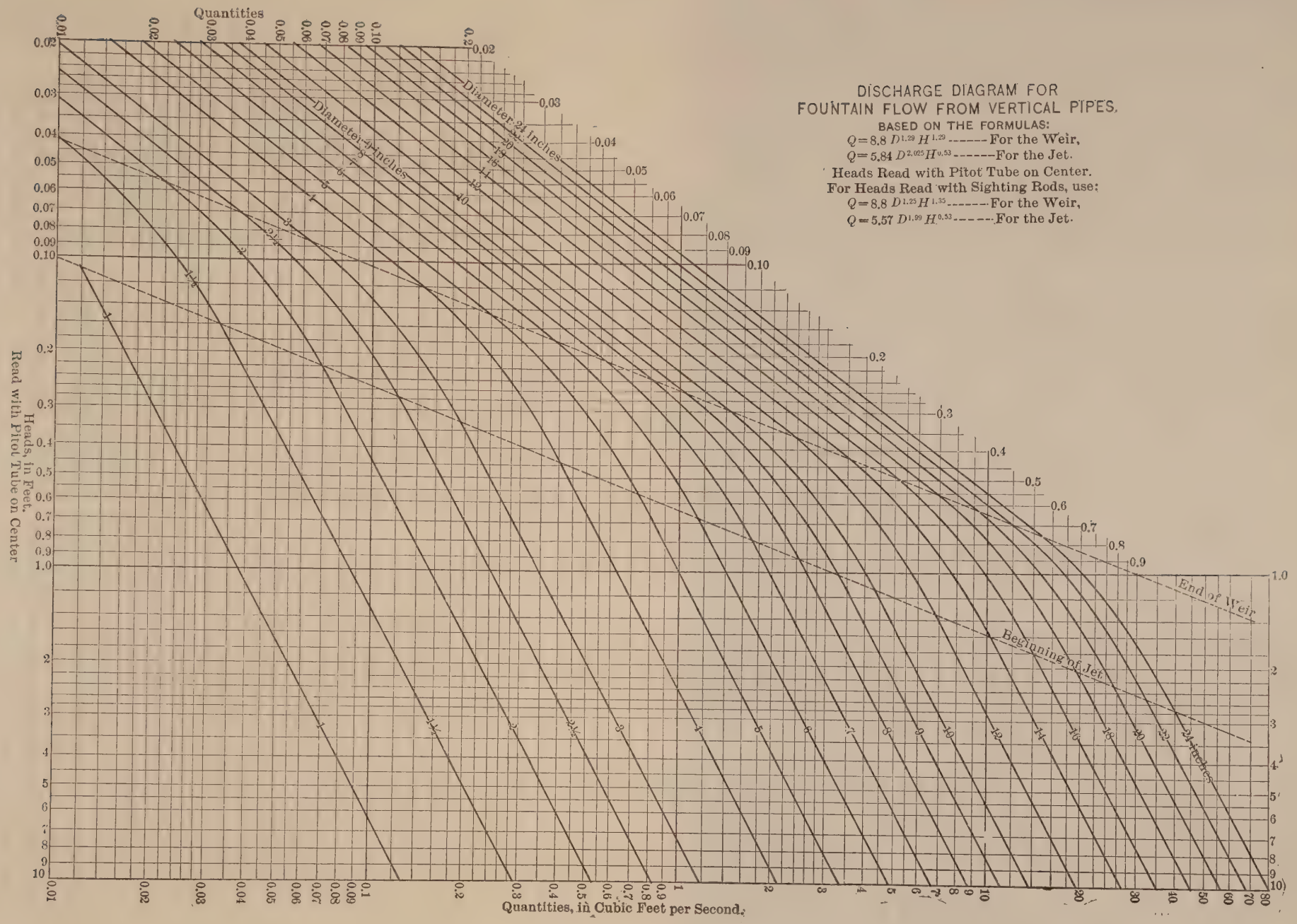
LAWRENCE AND BRAUNWORTH ON
FOUNTAIN FLOW OF WATER.



DISCHARGE DIAGRAM FOR FOUNTAIN FLOW FROM VERTICAL PIPES.

BASED ON THE FORMULAS:

$Q = 8.8\ D^{1.29}\ H^{1.29}$ ------ For the Weir,

$Q = 5.84\ D^{2.025}\ H^{0.53}$ ------ For the Jet.

Heads Read with Pitot Tube on Center.

For Heads Read with Sighting Rods, use:

$Q = 8.8\ D^{1.25}\ H^{1.35}$ ------ For the Weir,

$Q = 5.57\ D^{1.99}\ H^{0.53}$ ------ For the Jet.

TABLE 6.—(*Continued from page 298*).

Head, in feet.	Diameter of Pipe, in Inches.																		
	1	1½	2	2½	3	4	5	6	7	8	9	10	12	14	16	18	20	22	24
6.2	0.1000	0.2235	0.4080	0.6400	0.9260	1.655	2.598	3.770	5.160	6.76	8.60	10.62	15.32	21.10	26.81	35.00	43.40	52.40	62.60
6.4	0.1020	0.2303	0.4160	0.6510	0.9410	1.681	2.641	3.830	5.250	6.88	8.75	10.80	15.60	21.50	27.29	35.60	44.15	53.30	63.65
6.6	0.1035	0.2340	0.4225	0.6620	0.9580	1.710	2.682	3.900	5.335	7.00	8.90	11.00	15.82	21.81	27.73	36.20	44.80	54.10	64.70
6.8	0.1052	0.2379	0.4295	0.6720	0.9620	1.739	2.730	3.960	5.415	7.10	9.03	11.18	16.10	22.20	28.19	36.75	45.60	55.05	65.70
7.0	0.1066	0.2413	0.4355	0.6830	0.9870	1.761	2.768	4.020	5.500	7.205	9.18	11.32	16.34	22.52	28.60	37.30	46.25	55.90	66.70
7.2	0.1083	0.2455	0.4420	0.6930	1.005	1.790	2.810	4.080	5.590	7.315	9.32	11.50	16.60	22.86	29.08	37.90	47.00	56.80	67.80
7.4	0.1100	0.2486	0.4490	0.7020	1.020	1.815	2.853	4.400	5.670	7.42	9.46	11.64	16.82	23.20	29.46	38.45	47.70	57.55	68.80
7.6	0.1115	0.2523	0.4555	0.7130	1.032	1.841	2.890	4.200	5.750	7.53	9.58	11.85	17.06	23.53	29.90	39.00	48.35	58.35	69.90
7.8	0.1130	0.2560	0.4615	0.7230	1.046	1.868	2.935	4.260	5.820	7.63	9.72	12.00	17.32	23.85	30.35	39.55	49.00	59.15	70.70
8.0	0.1147	0.2590	0.4675	0.7320	1.060	1.892	2.970	4.315	5.905	7.73	9.84	12.18	17.55	24.19	30.70	40.05	49.70	60.00	71.60
8.2	0.1161	0.2627	0.4740	0.7430	1.074	1.920	3.015	4.375	5.990	7.84	9.99	12.33	17.78	24.50	31.15	40.60	50.35	60.80	72.50
8.4	0.1176	0.2660	0.4800	0.7520	1.086	1.941	3.050	4.430	6.060	7.95	10.10	12.49	18.00	24.80	31.60	41.10	51.00	61.55	73.50
8.6	0.1191	0.2695	0.4865	0.7620	1.100	1.966	3.085	4.485	6.140	8.04	10.23	12.65	18.22	25.12	31.95	41.65	51.60	62.30	74.40
8.8	0.1205	0.2730	0.4920	0.7710	1.113	1.993	3.130	4.540	6.210	8.14	10.36	12.80	18.45	25.42	32.30	42.15	52.30	63.10	75.30
9.0	0.1220	0.2760	0.4980	0.7800	1.126	2.015	3.160	4.590	6.290	8.23	10.47	12.95	18.66	25.73	32.70	42.65	52.90	63.90	76.20
9.2	0.1234	0.2792	0.5040	0.7900	1.140	2.040	3.200	4.650	6.360	8.33	10.60	13.11	18.90	26.00	33.05	43.20	53.50	64.60	77.10
9.4	0.1248	0.2825	0.5100	0.7990	1.152	2.062	3.240	4.700	6.430	8.43	10.73	13.25	19.10	26.35	33.50	43.70	54.20	65.30	78.00
9.6	0.1263	0.2860	0.5150	0.8070	1.166	2.085	3.275	4.760	6.510	8.52	10.85	13.41	19.32	26.62	33.80	44.15	54.80	66.05	78.90
9.8	0.1275	0.2886	0.5220	0.8180	1.179	2.107	3.315	4.810	6.590	8.62	10.98	13.56	19.55	26.93	34.20	44.65	55.30	66.90	79.90
10.0	0.1290	0.2920	0.5270	0.8250	1.190	2.130	3.350	4.860	6.640	8.70	11.07	13.69	19.79	27.20	34.60	45.10	56.00	67.60	80.55

TABLE 6.—(*Continued from page 299*).

Head, in feet.	Diameter of Pipe, in Inches.																		
	1	1⅛	2	2⅛	3	4	5	6	7	8	9	10	12	14	16	18	20	22	24
2.4	0.0605	0.1367	0.2469	0.3870	0.5600	1.0000	1.572	2.277	3.115	4.085	5.20	6.41	9.26	12.75	16.20	21.15	26.22	31.60	37.50
2.6	0.0632	0.1426	0.2576	0.4040	0.5830	1.0410	1.640	2.373	3.260	4.265	5.42	6.70	9.67	13.30	16.92	22.08	27.39	33.05	39.30
2.8	0.0657	0.1485	0.2680	0.4200	0.6080	1.082	1.701	2.470	3.380	4.44	5.64	6.97	10.02	13.84	17.60	23.00	28.45	34.40	41.00
3.0	0.0681	0.1540	0.2779	0.4350	0.6300	1.160	1.765	2.562	3.510	4.60	5.86	7.22	10.42	14.36	18.25	23.80	29.50	35.65	42.55
3.2	0.0704	0.1595	0.2876	0.4500	0.6520	1.163	1.830	2.651	3.630	4.765	6.05	7.48	10.80	14 85	18.86	24.61	30.60	36.90	44.00
3.4	0.0727	0.1645	0.2970	0.4655	0.6730	1.200	1.886	2.740	3.755	4.915	6.25	7.71	11.14	15.33	19.50	25.45	31.55	37.10	45.45
3.6	0.0750	0.1699	0.3065	0.4800	0.6950	1.240	1.947	2.825	3.870	5.07	6.45	7.95	11.48	15.81	20.08	26.21	32.50	39.25	46.85
3.8	0.0772	0.1746	0.3155	0.4945	0.7150	1.275	2.000	2.903	3.980	5.21	6.63	8.20	11.81	16.27	20.66	26.99	33.50	40.45	48.20
4.0	0.0794	0.1795	0.3240	0.5070	0.7340	1.325	2.060	2.985	4.085	5.36	6.81	8.41	12.15	16.72	21.25	27.70	34.35	41.55	49.60
4.2	0.0814	0.1841	0.3325	0.5210	0.7530	1.342	2.110	3.070	4.200	5.50	7.00	8.63	12.46	17.19	21.82	28.46	35.30	42.60	50.90
4.4	0.0836	0.1886	0.3410	0.5330	0.7720	1.380	2.165	3.140	4.300	5.63	7.18	8.86	12.80	17.60	22.39	29.20	36.15	43.70	52.10
4.6	0.0854	0.1932	0.3490	0.5470	0.7900	1.425	2.215	3.215	4.400	5.77	7.34	9.06	13.08	18.01	22.91	29.86	37.00	44.75	53.40
4.8	0.0873	0.1976	0.3570	0.5590	0.8080	1.441	2.270	3.290	4.505	5.90	7.51	9.27	13.40	18.42	23.42	30.55	37.85	45.95	54.60
5.0	0.0893	0.2020	0.3650	0.5710	0.8260	1.475	2.320	3.360	4.600	6.03	7.67	9.48	13.67	18.81	23.90	31.20	38.70	46.70	55.80
5.2	0.0912	0.2062	0.3720	0.5830	0.8430	1.505	2.365	3.430	4.700	6.17	7.83	9.69	13.99	19.22	24.44	31.90	39.55	47.75	57.00
5.4	0.0930	0.2103	0.3800	0.5950	0.8600	1.538	2.415	3.500	4.800	6.285	8.00	9.88	14.25	19.62	24.93	32.55	40.30	48.75	58.10
5.6	0.0949	0.2148	0.3870	0.6070	0.8780	1.566	2.463	3.575	4.890	6.405	8.15	10.06	14.52	20.00	25.42	33.15	41.10	49.70	59.30
5.8	0.0968	0.2185	0.3945	0.6190	0.8940	1.595	2.507	3.640	4.970	6.515	8.30	10.25	14.80	20.40	25.90	33.80	41.85	50.55	60.40
6.0	0.0983	0.2225	0.4015	0.6290	0.9100	1.625	2.553	3.705	5.065	6.64	8.45	10.43	15.05	20.55	26.39	34.40	42.60	51.50	61.40

TABLE 6.—(*Continued from page 300*).

Head, in feet.	Diameter of Pipe, in Inches.																		
	1	1½	2	2½	3	4	5	6	7	8	9	10	12	14	16	18	20	22	24
0.650	0.0303	0.0683	0.1238	0.1939	0.2801	0.5000	0.7850	1.138	1.547	2.00	2.468	3.06	4.22	5.50	6.86	8.26	9.70	11.00	12.20
0.700	0.0316	0.0711	0.1287	0.2015	0.2916	0.5195	0.8170	1.180	1.615	2.095	2.605	3.22	4.49	5.87	7.34	8.91	10.53	12.10	13.40
0.750	0.0328	0.0738	0.1333	0.2090	0.3025	0.5390	0.8460	1.227	1.680	2.183	2.738	3.38	4.73	6.25	7.80	9.50	11.40	13.11	14.80
0.800	0.0338	0.0762	0.1380	0.2161	0.3130	0.5575	0.8765	1.268	1.740	2.270	2.850	3.53	4.97	6.58	8.23	10.10	12.13	14.02	15.95
0.850	0.0349	0.0788	0.1425	0.2230	0.3230	0.5750	0.9050	1.310	1.800	2.352	2.96	3.67	5.19	6.90	8.65	10.68	12.88	14.86	17.07
0.900	0.0360	0.0812	0.1467	0.2300	0.3330	0.5925	0.9350	1.351	1.855	2.427	3.065	3.80	5.38	7.19	9.03	11.22	13.60	15.70	18.16
0.950	0.0370	0.0837	0.1512	0.2365	0.3425	0.6110	0.9600	1.390	1.907	2.500	3.165	3.92	5.56	7.45	9.39	11.75	14.27	16.53	19.20
1.00	0.0381	0.0860	0.1550	0.2433	0.3515	0.6270	0.9860	1.429	1.960	2.568	3.270	4.03	5.73	7.71	9.71	12.27	14.99	17.40	20.20
1.1	0.0400	0.0908	0.1634	0.2559	0.3705	0.6600	1.040	1.502	2.061	2.705	3.430	4.250	6.080	8.210	10.380	13.220	16.130	18.93	22.07
1.2	0.0419	0.0955	0.1710	0.2680	0.3880	0.6900	1.087	1.575	2.160	2.832	3.600	4.45	6.39	8.66	10.95	14.08	17.20	20.30	23.78
1.3	0.0438	0.0989	0.1785	0.2795	0.4050	0.7205	1.135	1.642	2.252	2.955	3.760	4.64	6.69	9.08	11.48	14.86	18.20	21.60	25.35
1.4	0.0455	0.1027	0.1859	0.2910	0.4210	0.7500	1.180	1.706	2.343	3.075	3.905	4.825	6.97	9.49	12.00	15.58	19.12	22.85	26.80
1.5	0.0472	0.1065	0.1925	0.3015	0.4365	0.7780	1.225	1.773	2.430	3.180	4.055	5.00	7.22	9.89	12.50	16.25	20.00	23.90	28.08
1.6	0.0489	0.1102	0.1995	0.3125	0.4520	0.8050	1.267	1.835	2.515	3.300	4.195	5.18	7.48	10.27	12.99	16.88	20.80	24.92	29.30
1.7	0.0504	0.1140	0.2060	0.3220	0.4670	0.8305	1.306	1.897	2.600	3.408	4.330	5.35	7.71	10.62	13.45	17.46	21.55	25.90	30.50
1.8	0.0520	0.1173	0.2120	0.3320	0.4810	0.8560	1.350	1.955	2.678	3.510	4.465	5.51	7.96	10.95	13.90	18.00	22.36	26.83	31.60
1.9	0.0535	0.1208	0.2181	0.3420	0.4950	0.8810	1.388	2.010	2.755	3.610	4.590	5.67	8.19	11.26	14.33	18.58	23.00	27.70	32.70
2.0	0.0550	0.1241	0.2243	0.3515	0.5080	0.9060	1.426	2.063	2.832	3.710	4.720	5.82	8.41	11.59	14.70	19.15	23.60	28.53	33.75
2.2	0.0578	0.1306	0.2360	0.3690	0.5340	0.9530	1.500	2.170	2.978	3.905	4.970	6.13	8.85	12.19	15.47	20.20	24.96	30.10	35.60

TABLE 6.—(*Continued from page 301*).

Head, in feet.	Diameter of Pipe, in Inches.																		
	1	1⅛	2	2½	3	4	5	6	7	8	9	10	12	14	16	18	20	22	24
0.100	0.0112	0.0238	0.0392	0.0564	0.0740	0.1140	0.1410	0.1805	0.2220	0.262	0.300	0.352	0.439	0.539	0.638	0.731	0.849	0.980	1.08
0.110	0.0118	0.0255	0.0427	0.0623	0.0823	0.1290	0.1510	0.2050	0.2510	0.298	0.341	0.400	0.500	0.611	0.721	0.832	0.967	1.11	1.23
0.120	0.0124	0.0270	0.0458	0.0677	0.0905	0.1440	0.1800	0.2290	0.2810	0.332	0.381	0.447	0.558	0.682	0.805	0.937	1.10	1.24	1.37
0.130	0.0129	0.0284	0.0489	0.0723	0.0981	0.1555	0.2000	0.2550	0.3120	0.372	0.422	0.495	0.619	0.760	0.903	1.06	1.22	1.38	1.52
0.140	0.0134	0.0299	0.0517	0.0769	0.1055	0.1715	0.2210	0.2800	0.3430	0.406	0.463	0.543	0.681	0.839	1.00	1.16	1.34	1.51	1.68
0.150	0.0139	0.0312	0.0544	0.0810	0.1123	0.1841	0.2410	0.3070	0.3750	0.442	0.508	0.599	0.742	0.920	1.08	1.28	1.47	1.66	1.84
0.160	0.0144	0.0324	0.0569	0.0853	0.1185	0.1965	0.2610	0.3330	0.4080	0.483	0.552	0.648	0.811	1.00	1.18	1.38	1.59	1.80	2.00
0.170	0.0148	0.0336	0.0592	0.0892	0.1245	0.2090	0.2810	0.3600	0.4420	0.521	0.600	0.701	0.881	1.07	1.28	1.49	1.72	1.94	2.16
0.180	0.0153	0.0346	0.0613	0.0930	0.1302	0.2208	0.3000	0.3860	0.4750	0.561	0.642	0.753	0.950	1.16	1.38	1.60	1.85	2.09	2.33
0.190	0.0158	0.0356	0.0633	0.0966	0.1359	0.2323	0.3180	0.4120	0.5100	0.600	0.683	0.805	1.02	1.24	1.48	1.73	1.99	2.25	2.49
0.200	0.0162	0.0365	0.0654	0.1000	0.1407	0.2427	0.3360	0.4380	0.5460	0.645	0.733	0.867	1.08	1.325	1.58	1.87	2.12	2.40	2.66
0.250	0.0182	0.0417	0.0746	0.1150	0.1642	0.2880	0.4150	0.5600	0.7080	0.857	0.990	1.17	1.44	1.78	2.11	2.49	2.84	3.20	3.52
0.300	0.0201	0.0454	0.0821	0.1285	0.1839	0.3250	0.4800	0.6620	0.8570	1.032	1.22	1.47	1.81	2.22	2.64	3.12	3.60	4.08	4.45
0.350	0.0218	0.0497	0.0892	0.1395	0.2021	0.3580	0.5390	0.7500	0.9900	1.22	1.43	1.75	2.22	2.72	3.23	3.81	4.40	4.97	5.45
0.400	0.0234	0.0529	0.0957	0.1495	0.2170	0.3855	0.5900	0.8320	1.081	1.362	1.63	2.02	2.66	3.24	3.84	4.50	5.20	4.90	6.50
0.450	0.0250	0.0562	0.1019	0.1591	0.2310	0.4105	0.6360	0.9180	1.190	1.51	1.816	2.26	3.01	3.79	4.49	5.16	6.09	5.85	7.58
0.500	0.0264	0.0595	0.1078	0.1683	0.2440	0.4350	0.6770	0.9750	1.291	1.646	1.99	2.47	3.35	4.24	5.08	5.98	6.95	6.82	8.62
0.550	0.0278	0.0626	0.1131	0.1772	0.2566	0.4570	0.7170	1.030	1.385	1.773	2.155	2.68	3.66	4.69	5.75	6.80	7.90	8.84	9.86
0.600	0.0291	0.0655	0.1185	0.1855	0.2695	0.4785	0.7520	1.085	1.468	1.89	2.313	2.87	3.90	5.11	6.30	7.57	8.80	9.90	11.00

TABLE 6.—Discharge, in Cubic Feet per Second, for Fountain Flow from Vertical Pipes.

Heads from 0.02 to 10 ft.

Head Observed by Pitot Tube on Center.

From Formulas 1 and 2 and Intermediate Condition.

{ Weir condition above single dotted line.
Intermediate condition between the two dotted lines.
Jet condition below dotted double line.

Head, in feet.	Diameter of Pipe, in Inches.																		
	1	1½	2	2½	3	4	5	6	7	8	9	10	12	14	16	18	20	22	24
0.020	.....	.....	.....	.....	0.0095	0.0142	0.0179	0.0226	0.0278	0.0330	0.0381	0.0443	0.0552	0.0688	0.0802	0.0920	0.1080	0.1240	0.1340
0.025	.....	.....	.....	0.0096	0.0125	0.0189	0.0236	0.0301	0.0371	0.0440	0.0501	0.0588	0.0737	0.0913	0.1070	0.1210	0.1430	0.1640	0.1780
0.030	.....	.....	0.0093	0.0125	0.0158	0.0240	0.0296	0.0382	0.0470	0.0550	0.0630	0.0735	0.0920	0.1140	0.1340	0.1550	0.1790	0.2080	0.2230
0.035	.....	.....	0.0112	0.0153	0.0194	0.0293	0.0362	0.0466	0.0573	0.0674	0.0775	0.0905	0.1120	0.1390	0.1640	0.1890	0.2180	0.2540	0.2720
0.040	.....	.....	0.0136	0.0181	0.0230	0.0348	0.0432	0.0555	0.0680	0.0800	0.0919	0.1070	0.1330	0.1660	0.1940	0.2230	0.2580	0.3010	0.3240
0.045	.....	0.0108	0.0158	0.0212	0.0269	0.0406	0.0501	0.0644	0.0790	0.0932	0.1070	0.1250	0.1550	0.1920	0.2240	0.2620	0.3010	0.3510	0.3780
0.050	.....	0.0122	0.0182	0.0242	0.0308	0.0465	0.0574	0.0741	0.0905	0.1060	0.1220	0.1420	0.1770	0.2200	0.2590	0.2980	0.3430	0.4020	0.4310
0.055	.....	0.0136	0.0205	0.0275	0.0349	0.0526	0.0650	0.0836	0.1030	0.1210	0.1380	0.1620	0.2010	0.2500	0.2940	0.338	0.391	0.457	0.491
0.060	.....	0.0149	0.0229	0.0307	0.0390	0.0590	0.0732	0.0935	0.1150	0.136	0.156	0.182	0.227	0.280	0.330	0.380	0.467	0.507	0.549
0.065	.....	0.0162	0.0251	0.0340	0.0433	0.0650	0.0800	0.1040	0.1278	0.151	0.172	0.201	0.250	0.310	0.364	0.422	0.484	0.563	0.610
0.070	.....	0.0174	0.0274	0.0375	0.0475	0.0720	0.0893	0.1141	0.1401	0.166	0.189	0.221	0.276	0.340	0.402	0.464	0.535	0.620	0.669
0.075	.....	0.0186	0.0295	0.0408	0.0520	0.0785	0.0980	0.1250	0.1518	0.181	0.203	0.243	0.303	0.375	0.440	0.504	0.583	0.680	0.717
0.080	0.0097	0.0198	0.0317	0.0438	0.0567	0.0855	0.1070	0.1360	0.1665	0.197	0.226	0.265	0.330	0.406	0.480	0.550	0.631	0.735	0.802
0.085	0.0102	0.0209	0.0337	0.0472	0.0608	0.0925	0.1150	0.1470	0.1800	0.216	0.244	0.287	0.357	0.440	0.520	0.595	0.688	0.796	0.870
0.090	0.0105	0.0219	0.0356	0.0504	0.0654	0.0995	0.1230	0.1580	0.1940	0.230	0.262	0.310	0.384	0.473	0.559	0.642	0.741	0.854	0.940
0.095	0.0108	0.0229	0.0376	0.0534	0.0698	0.1067	0.1330	0.1700	0.2080	0.245	0.281	0.330	0.412	0.506	0.600	0.684	0.794	0.920	1.008

DISCUSSION.

M. A. Flamant. A. FLAMANT, ESQ.,* (by letter).—The writer has examined this paper with great interest, and cannot congratulate the authors too highly for the great care with which they have conducted their experiments, and the ingenious precautions they have devised in order to eliminate all causes of error.

In view of the particular question which they have investigated, their work is valuable, and the results will certainly form a basis for much theoretical study.

At this time, the writer has not the leisure to devote himself to this study, but, from a single reading of the paper, he thinks that the results of these experiments seem to deviate very little from those given by certain formulas with very simple exponents, such as $D^{\frac{5}{4}} H^{\frac{5}{4}}$ for weir-flow, and $D^2 H^{\frac{1}{2}}$ for jet-flow formulas, the correctness of which, without doubt, would be easily proved by abstract theory. The differences are such as are admitted, ordinarily, in questions in which absolute accuracy is seldom attempted. The writer regrets greatly that he is not able, at this time, to make a comparison of these formulas and observations, which would perhaps be instructive.

Mr. Thrupp. EDGAR C. THRUPP, ESQ.,† (by letter).—This paper provides a useful addition to the data for gauging water under certain conditions without resorting to expensive appliances.

The point which rather limits its convenience is the necessity of having 10 or 15 ft. of vertical pipe below the outlet to establish uniform vertical flow. This difficulty does not arise in the case of discharges from Artesian wells, or inlets to service reservoirs, etc., which are usually more than 15 ft. deep, and it would often be easy to adopt the system on the supply pipes to sewage and water-supply filter beds. The form of the outlet is quite simple, and is easily reproduced with accuracy, which is an essential feature with gauging appliances.

Other instances where further experimental study might lead to useful results might be mentioned, for example, the border line between weir flow and orifice flow in the case of discharge through large rectangular, vertical orifices which are not always submerged.

Another matter, in connection with experimental studies of this kind, is the limitation imposed by having to measure the whole flow into a tank which must have (for accurate work) a capacity of 2 min. flow, or more. This generally fixes the largest flow used, but the writer has found by experience that it is not difficult to construct a weir which, although unsuitable for exact gauging purposes by itself, serves to spread out the stream in so uniform a man-

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† Associate Member, Institution of Civil Engineers.

ner that an exactly known proportion can be diverted into a measuring tank through a trough. By this means the scope of experiments can be largely increased without requiring huge measuring tanks or weirs with large approach channels which take a long time to settle down to uniform conditions of flow for each observation. Experimenters will soon appreciate the saving of time and expense rendered possible by this plan. Mr. Thrupp.

FRANK L. GETMAN, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has often found himself in need of some simple method for measuring the flow of Artesian wells which would be applicable in the field and give fairly accurate results. Mr. Getman.

In 1903 when engaged in sinking Artesian wells at Ithaca, N. Y., the writer had at hand a good weir box, to measure the discharge of the wells, and took measurements of the head over the edge of the pipe in order to check the results by the table prepared by Professor G. E. Todd, but it was found that the quantities shown by that table were worse than useless. Desiring to find a formula by which the discharge from a vertical pipe could be computed, the head over the edge being known, the writer made inquiries at the hydraulic laboratory of Cornell University, but was informed that, with the exception of the work by Professor Todd, little was known on the subject. I. P. Church, Assoc. Am. Soc. C. E., however, suggested that with low heads, when the water left the edge of the pipe in a smooth sheet, the pipe might be considered as a circular weir discharging under a head equal to the head over the edge of the pipe measured at the center. Further inquiry, through a letter published in *Engineering News,* failed to elicit any information except in reference to Professor Todd's tables, one for heads on a vertical pipe, and one for wells discharging through a horizontal pipe by measuring vertical and horizontal co-ordinates from the end of the pipe to the center of the discharged stream.

The writer's measurements were limited to six 6-in. wells, with heads varying from $\frac{1}{16}$ to $2\frac{3}{4}$ in., the first four of which came under the weir condition, and the fifth and sixth under the intermediate condition, as deduced by Messrs. Lawrence and Braunworth. In Well No. 1 the flow was so light that the water sheet clung to the sides of the pipe, giving a condition which was not a weir flow.

The daily delivery of these wells ranged from about 40 000 to more than 300 000 gal. per day, presenting a fair field for comparative results.

The head on the pipe was measured by holding a saw blade or sheet of strong paper—always obtainable in the field—diametrically across the pipe in the water column, and noting the maximum height marked by the water. The discharge of the well was then

Mr. Getman. led into the weir box, which was a better one than is usually obtainable for field work, being 12 ft. long and 4 ft. wide, and provided with two baffle boards and a 12-in. rectangular weir with steel knife-edges.

TABLE 7.—COMPARISON OF DIFFERENT METHODS OF COMPUTING THE FLOW FROM ARTESIAN WELLS.

Well No.	Diameter of well in inches.	Depth on Edge of Pipe.		Depth on 1-ft. Weir.		Discharge, in Cubic Feet per Second, Computed by Different Methods.				
		Inches.	Feet.	Inches.	Feet.	(I) By Weir.	(II) By Todd's Table.	(III) By considering well pipe as a circular weir.	(IV) By $Q = 8.8\ D^{1.29}\ H^{1.29}$	(V) By $Q = 8.8\ D^{1.25}\ H^{1.35}$
1	6	$1\frac{1}{16}$	0.0573	$\frac{7}{8}$	0.0729	0.06577	0.3663	0.07184	0.0882	0.07811
2	6	$1\frac{3}{4}$	0.0625	$1\frac{1}{16}$	0.0885	0.08775	0.3830	0.08203	0.0987	0.09389
3	6	$1\frac{13}{16}$	0.0677	$1\frac{1}{8}$	0.0938	0.09570	0.3995	0.09263	0.10945	0.09761
4	6	2	0.1667	$2\frac{3}{16}$	0.1979	0.29338	0.6433	0.35544	0.3511	0.33216
5	6	$2\frac{3}{8}$	0.1979	$2\frac{5}{8}$	0.2396	0.39100	0.6965	0.46063	0.43254	0.41540
6	6	$2\frac{3}{4}$	0.2292	$3\frac{5}{15}$	0.2760	0.48260	0.8330	0.57330	0.5112	0.50625

On Well No. 1 the water clung to the sides of the pipe; on all other wells the sheet flowed free.

TABLE 8.—ERROR, IN METHODS OF COMPUTING THE FLOW FROM ARTESIAN WELLS, EXPRESSED AS PERCENTAGES OF THE FLOW SHOWN BY THE WEIR.

Well No.	(I) By Weir.	(II) By Todd's Table.	(III) By considering pipe as a circular weir.	(IV) By $Q = 8.8\ D^{1.29}\ H^{1.29}$	(V) By $Q = 8.8\ D^{1.25}\ H^{1.35}$
1	0.00%	+457%	+ 9.2%	+34.3%	+18.5%
2	0.00%	+337%	− 6.5%	+12.6%	+ 7.0%
3	0.00%	+314%	− 3.1%	+ 3.9%	+ 2.0%
4	0.00%	+119%	+21.2%	+19.6%	+18.2%
5	0.00%	+ 78%	+17.8%	+11.3%	+ 6.2%
6	0.00%	+ 73%	+18.8%	+ 5.9%	+ 4.9%

Tables 7 and 8 show the discharge of the six wells, in cubic feet per second, computed by the different available methods. In Table 7, Column I gives the delivery as shown by the weir, which would be satisfactory for measurements if it were always available and if the apparatus were less cumbersome for field observations; Column II gives the quantities deduced from Professor

Mr. Getman.

Todd's table; Column III gives the delivery computed by considering the edge of the 6-in. pipe as a weir with a length equal to the circumference, 1.57 ft.; Column IV gives the quantities taken by interpolation from the authors' Table 6, which is based on the formula $Q = 8.8\ D^{1.29}\ H^{1.29}$, for heads measured with a Pitot tube; and Column V gives the quantities computed from the formula, $Q = 8.8\ D^{1.25}\ H^{1.35}$, which was adapted to heads taken by rod readings, probably resembling the way in which the writer measured the heads on the wells.

Table 8 shows the errors in the different methods, expressed in percentages of the flow as indicated by the weir measurements, it being assumed that this is a sufficiently accurate method to be adopted as a standard. By Professor Todd's tables, the results range in error from + 457% for the lowest head to + 73% for the highest head. His tables contain data only for pipes up to 3 in. in diameter, but a statement is made that for larger pipes the flow is proportional to the square of the diameter, or to the area. Probably a part of the error is due to this supposition for larger pipes, but the fact remains that the method is so erroneous that the writer must preclude it from use for all heads.

By considering the pipe as a circular weir, the errors range from — 3 to + 21 per cent. This is a very simple method, and is available for field use, as Francis' weir formula or tables are always at hand. The percentages, for the heads considered, compare well with those by other methods, but, of course, would be applicable only to the weir condition.

In the formula for heads measured with the Pitot tube, although this formula is not strictly applicable for heads observed by direct measurements, the errors are interesting, as they range from + 34 to + 4%; while the errors for the formula designed for heads measured by rod readings, the best method for use with these heads as observed, vary from + 2 to + 18 per cent.

Of course, the writer's observations were limited and rough, compared with the careful and extensive work of Messrs. Lawrence and Braunworth, but they may be of interest in judging the value of the authors' formula as applied by those who utilize it in the field.

Mr. Schoder.

Ernest W. Schoder, Jun. Am. Soc. C. E. (by letter).—The writer feels that this paper is of value, not only because of its definite and comprehensive information, so well presented, but also because of its suggestiveness of other investigations of benefit to hydraulic science.

In practice there arises at times the desirability of providing a measuring weir with a very short approach channel, on account of restricted space horizontally. Usually, in such cases, there is ample room vertically.

Mr. Schoder.

The question comes: Why not let the water approach the weir crest vertically instead of horizontally, as is the case where a long approach channel is provided?

The usual weir formulas probably would not apply. A series of experiments to determine the laws of flow is necessary, in order to take the matter out of the realm of speculation. The effects of various arrangements of baffle partitions should be studied.

In the authors' paper it appears that for vertical pipes with weir flow the discharge varies about as the 1.3 power of the head. This is to be compared with the 1.5 power that applies to weirs with horizontal approach. Some analogous modification may be expected in cases where the water approaches a straight, level, weir crest vertically.

Messrs. Lawrence and Braunworth.

MESSRS. F. E. LAWRENCE AND P. L. BRAUNWORTH (by letter).—In regard to the percentage errors computed by Mr. Getman, it will be noted that all his results are too high. Of course, Well No. 1, in Mr. Getman's experiments, would have to be excluded because the flow with clinging discharge is subject to different laws from free sheet weir flow. In his experiments the heads were read by noting the maximum height reached on a sheet of paper or saw blade. As the writers have pointed out before, the maximum height should not be taken as the head over the pipe, because the water reaches that height, stays there a comparatively short time, and then drops. The time element must be considered in reading a head, that is, the head selected should be the one at which the water stays the longest time, thus representing an average head.

Supposing that the heads in Mr. Getman's experiments are too high, it can be seen that the percentage errors in his results would necessarily be too high. All these errors are positive, and a reduction in each would bring the average well within practical limits.

For field work, in the absence of a carefully made and finished Pitot tube, and when the heads are too low to admit of accurate sighting, the writers would recommend constructing a tube from any available piping. The end of the tube could be closed and a small hole drilled in the side near the end. A piece of rubber tubing fastened to the other end of the pipe would answer for a gauge simply by raising the end until the water just reached the top, and noting the difference in elevation between the pipe crest and the top of the tube.

In conclusion, the writers wish to express their appreciation for the interest taken in the subject and for the valuable suggestions contained in the discussions.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1036.

WORKS FOR THE PURIFICATION OF THE WATER SUPPLY OF WASHINGTON, D. C.*

By Allen Hazen and E. D. Hardy, Members, Am. Soc. C. E.

With Discussions by Messrs. Francis F. Longley, William C. Woodward, George M. Kober, Theodore A. Leisen, W. L. Butcher, J. P. A. Maignen, D. C. Henny, J. A. Vogleson, Charles Moore, Nora Stanton Blatch, William T. Sedgwick, H. F. Labelle, G. Lloyd Magruder, George W. Fuller, William B. Fuller, Theodore Horton, John H. Gregory, Allen Hazen and E. D. Hardy.

A sand filtration plant for the purification of the public water supply of Washington, D. C., has recently been put in service, and at present about 70 000 000 gal. of water per day are being treated.

The following is an account of the steps leading up to the construction of this plant, and a description of some of the more important and novel features of its construction and operation.

General Arrangement of the Water-Works.

For the last fifty years the water supply of Washington has been obtained from the Potomac River at Great Falls, about 14 miles above the city. At this point the water has a low-water elevation of 150.5 ft. above mean tide.

* Presented at the meeting of November 7th, 1906.

Aqueduct.—The water flows through an aqueduct with a slope of 0.00015 (or 9½ in. per mile), and is 9 ft. in diameter, but some unlined tunnel sections through hard rock are larger. There are many interesting structures on the line of the aqueduct, among them the Cabin John Bridge, until recently the longest masonry arch in existence.

The water passes through two reservoirs before reaching its present terminus in the Washington City Reservoir. These two reservoirs have by-passes, but usually all the water flows through them. The aqueduct is under slight pressure, the level of the water in it being as much as 2 or 3 ft. above its top; the hydraulic slope, therefore, usually differs from, and is less than, the actual slope of the masonry. It must have been the original intention to use the aqueduct in this way, for the flow lines of the reservoirs are some feet above the top of the aqueduct.

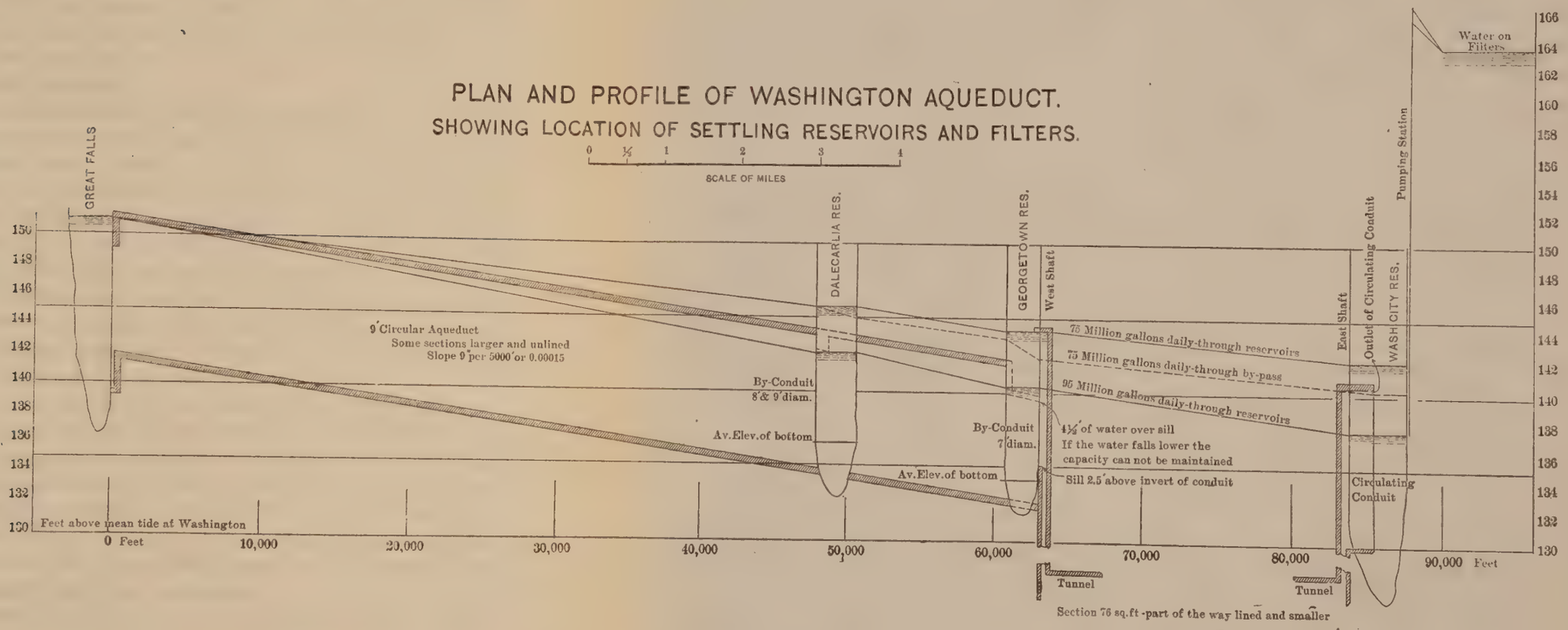
Reservoirs.—The first reservoir, 9 miles below Great Falls, known as the Dalecarlia Reservoir, was formed by damming a natural valley. It is irregular in shape, and comparatively shallow. The second, or Georgetown Reservoir, 12 miles from Great Falls, is built with embankments on a hillside, on a rectangular plan, and is of fair depth. The city was supplied for a long time by pipes leading directly from the Georgetown Reservoir. The aqueduct and these two reservoirs were constructed by General Meigs forty-five years ago.

More recently, a tunnel has been constructed from the Georgetown Reservoir to a central point in the city near Howard University, and a third reservoir, called the Washington City Reservoir, has been built at its terminus. This tunnel is 20 756 ft. long, generally 9 ft. in diameter, lined throughout, and is in the shape of a deep inverted siphon. Under Rock Creek it is 170 ft. below the hydraulic grade, and therefore under corresponding pressure. This portion has an inner lining of flanged, cast-iron plates, filled in with concrete, and is smaller in diameter.

The capacities of the three reservoirs, from their bottoms to their nominal flow lines, are approximately 150 000 000, 150 000 000 and 300 000 000 gal., respectively, making a total storage capacity of 600 000 000 gal. As the reservoirs cannot be filled to their flow lines when the full capacity of the aqueduct is used, and cannot

PLAN AND PROFILE OF WASHINGTON AQUEDUCT.

SHOWING LOCATION OF SETTLING RESERVOIRS AND FILTERS.


0 ¼ 1 2 3 4
SCALE OF MILES
GREAT FALLS
DALECARLIA RES.
GEORGETOWN RES.
West Shaft
East Shaft
Outlet of Circulating Conduit
Pumping Station
WASH. CITY RES.
Water on Filters
9' Circular Aqueduct
Some sections larger and unlined
Slope 9" per 5000' or 0.00015
By-Conduit 8' & 9' diam.
Av. Elev. of bottom
By-Conduit 7' diam.
Av. Elev. of bottom
75 Million gallons daily-through reservoirs
75 Million gallons daily-through by-pass
95 Million gallons daily-through reservoirs
4½' of water over sill
If the water falls lower the capacity can not be maintained
Sill 2.5' above invert of conduit
Circulating Conduit
Tunnel
Tunnel
Section 76 sq. ft - part of the way lined and smaller
Feet above mean tide at Washington
150 148 146 144 142 140 138 136 134 132 130
166 164 162 160 158 156 154 152 150 148 146 144 142 140 138 136 134 132 130
0 Feet 10,000 20,000 30,000 40,000 50,000 60,000 70,000 80,000 90,000 Feet

be drawn to their bottoms, only a part of this, roughly estimated at 300 000 000 gal., can be utilized.

Capacity of the Aqueduct.—The most reliable information as to the capacity of the aqueduct from Great Falls to the city was obtained by a series of current-meter observations made by Captain (now Major) Gaillard in 1897, after raising the dam at Great Falls and thoroughly cleaning the aqueduct. He found that with the water at Great Falls at the crest of the dam, as reconstructed at Elevation 150.5, and with the water in the Georgetown Reservoir at 144, or somewhat below the level at which it has been usually carried, water would flow through the aqueduct at the rate of 76 500 000 gal. per day.

When the difference in the water levels is less, that is to say, when the water in the distributing reservoir is higher, the carrying capacity of the aqueduct is reduced; but, on the other hand, with the reservoirs in the city drawn below their usual levels, and with the water at Great Falls higher, the carrying capacity is considerably increased.

For practical purposes it has been considered that 75 000 000 gal. represents the full carrying capacity of the aqueduct, as this is the largest quantity which can be drawn without reducing the water levels to a greater extent than has been thought advisable.

Since the filters have been in operation the quantity of water passing through the aqueduct has been known with accuracy, and from these records the resistances of the different parts of the aqueduct and tunnel are known with greater accuracy. The results obtained check substantially Major Gaillard's work and the further computations based upon it which have been used for estimating the discharge in the interval.

High and Low Service.—The greater part of Washington was supplied with water from the Georgetown and Washington City Reservoirs by gravity up to the time when the filters were put in service. The pressures obtained in this way were less than usually considered desirable in modern cities. Most of the buildings in Washington, however, are comparatively low, and, with ample pipe capacities, a reasonable supply for fire-engines and for domestic purposes has been maintained.

The higher portions of the city are supplied from two high-

service reservoirs. Water is pumped to these from the low-service pipes. A pumping station in U Street was used for this purpose for many years, and has recently been superseded by the new and ample pumping station at Trumbull Street, near the Washington City Reservoir.

Quality of the River Water.—The Potomac River at Great Falls has a drainage area of approximately 11 043 sq. miles. The most important cities and towns upon its water-shed, with their populations for the last three censuses, and the rural populations, are given in Table 1.

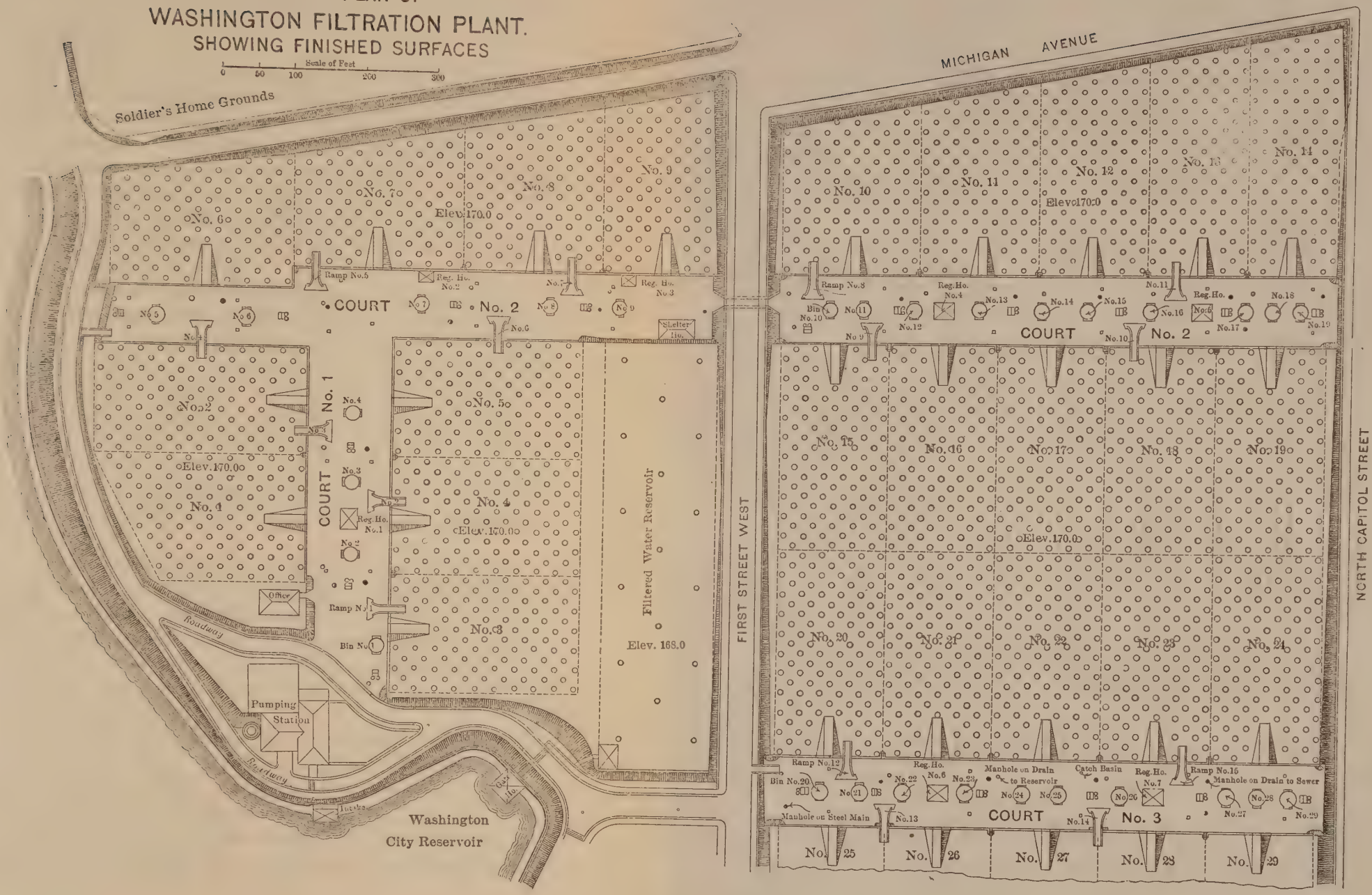
TABLE 1.—POPULATION ON THE WATER-SHED OF THE POTOMAC RIVER.

Drainage area at Great Falls = 11 043 sq. miles.

Place.	County.	State.	Approximate distance above Great Falls, in miles.	Population.		
				1880.	1890.	1900.
Urban: Places of more than 4 000:						
Frederick	Frederick	Md.	45	8 659	8 193	9 296
Hagerstown	Washington	Md.	70	6 627	10 118	13 591
Martinsburg	Berkeley	W. Va.	75	6 335	7 226	7 564
Waynesboro	Franklin	Pa.	85	1 888	3 811	5 396
Winchester	Frederick	Va.	100	4 958	5 196	5 161
Chambersburg	Franklin	Pa.	115	6 877	7 863	8 864
Cumberland	Allegheny	Md.	155	16 693	12 729	17 128
Frostburg	Allegheny	Md.	165	.....	3 804	5 274
Staunton	Augusta	Va.	175	6 664	6 975	7 289
Total population, towns having more than 4 000 inhabitants				52 701	65 915	79 563
Places having from 1 000 to 4 000 inhabitants				23 087	27 153	37 523
Rural population (being all the remainder)				384 664	405 155	420 007
Total population upon water-shed				460 452	498 223	537 093
Population per square mile, places having more than 4 000 inhabitants in 1900				4.7	5.9	7.2
Places having from 1 000 to 4 000 inhabitants in 1900				2.1	2.4	3.4
Rural population per square mile				34.9	36.8	38.0
Total population per square mile				41.7	45.1	48.6
Increase in 10 years				.....	8.1%	7.7%

The total population, and especially the urban population, is small in comparison with that upon many water-sheds used with filtration for public water supplies. The sewage pollution of the

GENERAL PLAN OF WASHINGTON FILTRATION PLANT. SHOWING FINISHED SURFACES



water, therefore, is not great in total amount, but the conditions seem to have been favorable for the transmission of infection, for the typhoid rate in Washington has been high, and at times, at least, has followed the prevalence of the disease in the towns upon the water-shed.

The Potomac water is rather muddy. The amount of sediment carried by it, on an average, is many times greater than that carried by the rivers of New England and of the sandy glaciated regions of the Northern United States. On the other hand, the suspended matters are much less in amount than in most southern streams. Probably, on an average, they do not amount to more than one-fourth of the quantities carried by the Ohio River at Louisville, or to more than one-twelfth of those carried by the Missouri at its mouth above St. Louis.

The amount of suspended matter carried by the water fluctuates widely. There are periods, sometimes lasting for months, when the Potomac water is comparatively clear. At other times the water is excessively turbid, and at times very turbid water flows continuously in the river for a week or more. At these times the water may approach the condition of the Ohio and Missouri waters, although the duration of the turbid periods is less.

The muddiness of the Potomac water and its pollution by sewage, and consequently its disease-producing qualities, have alike contributed to its bad reputation and to a desire for an improved water supply.

Investigations for Filtration.—As early as 1894 the late Colonel George H. Elliott proposed the filtration of the Potomac River water by mechanical filters, and made a rough estimate of cost. In 1899 an appropriation of $3 000, afterward increased to $8 000, was made, to investigate experimentally the filtration of the Potomac water, and to cover the expense of making a report upon its advantages and cost.

Acting under this authority, the late Lieutenant-Colonel A. M. Miller, M. Am. Soc. C. E., then in charge of the Washington Aqueduct, organized and conducted a series of experiments with sand and mechanical filters, examined possible sites, and made preliminary plans and estimates of cost. In conducting these experiments Colonel Miller was assisted in the experimental work

by R. S. Weston, Assoc. M. Am. Soc. C. E., while Mr. Hardy, one of the writers, also investigated the possible locations for filters, and made preliminary designs and estimates of cost.

Colonel Miller's investigation was the first to determine accurately and adequately the character of the Potomac water with reference to the possibilities of its purification; and, among many other interesting results, served to demonstrate: first, that the water could not be filtered successfully by sand filters without the use of a coagulant; and second, that the water could be coagulated readily and successfully, and that when coagulated satisfactory effluents could be obtained from it with mechanical filters. Colonel Miller recommended the establishment of mechanical filters.

The matter being before Congress, a protest was made by the Medical Society of the District of Columbia. The doctors were opposed to the use of a coagulant, and believed that better and more certain results, particularly from a hygienic standpoint, could be obtained by the use of sand filters without coagulant. The contention of the Medical Society seemed to be irreconcilable with Colonel Miller's findings.

To obtain further information upon the points at issue, the Senate Committee on the District of Columbia, of which the late James McMillan was Chairman, gave a hearing in New York City, at which many persons interested or experienced in filtration were present, and were questioned as to the various matters.

Following this, the matter was referred by the Senate Committee to Rudolph Hering, George W. Fuller and Allen Hazen, Members, Am. Soc. C. E., for report as to the best method to be followed. This engineering committee had but a short time for its work, as Congress was about to adjourn, and immediate action was desired. The ground was canvassed by the committee rapidly, and a recommendation made that sand filters should be used, but that the Potomac water should receive a preliminary chemical treatment at those times when it was of such a character that after sedimentation alone it was incapable of yielding a satisfactory effluent. This recommendation thus met substantially the contentions of both parties. It was estimated from the records of the Washington Aqueduct that it would be necessary to treat the water in this way, on an average, about 30 days of the year, and that

PLATE XXXII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVII, No. 1036.
HAZEN AND HARDY ON
WATER FILTRATION
AT WASHINGTON, D. C.

FIG. 1.—FLOOR BLOCKS AND INVERTED GROINED ARCHES.

FIG. 2.—INTERIOR, FILTERED WATER RESERVOIR.

during the remainder of the time the water could be filtered without such treatment.

This recommendation was adopted by the Senate Committee, but, as the doctors still objected to the use of a coagulant, the appropriation was made for the construction of sand filters, without authority for its use. This was done with the understanding that the appliances for using a coagulant could be put in equally well at a later date, and that failure to authorize them at that time would not delay the introduction of filtered water.

Selection of Site for Filters.—After Congress had appropriated the money for the construction of sand filters, a Board, composed of Colonel Miller, Major Beach and Major Gaillard, was appointed by Brigadier-General John M. Wilson, Chief of Engineers, U. S. Army, to select a site. One site was considered near the line of the aqueduct and between it and the Potomac River, about 3 miles below Great Falls. A second site was considered near Brightwood Avenue, adjacent to and north of the Washington City Reservoir. The third site considered was between the Washington City Reservoir and North Capitol Street and between the Soldiers' Home Grounds and Cincinnati Street. This last site was selected. It was by far the most difficult and expensive site upon which to build, but it seemed to utilize more fully the existing plant, and was, moreover, in a central location.

Preparation of Plans.—Plans for filters upon this site were undertaken by Colonel Miller, assisted by Lieutenant (now Captain) George M. Hoffman, and by one of the writers, Mr. Hardy. Considerable progress was made, and a set of plans was prepared. Afterward, further consideration of certain matters led to the withdrawal of these plans, and the preparation of a revised set differing somewhat from the earlier ones. Both the writers were engaged upon these revised plans, under the direction of Colonel Miller, and assisted by Lieutenant (now Captain) W. P. Wooten, Assoc. M. Am. Soc. C. E., by R. D. Chase, Assoc. M. Am. Soc. C. E. (from Mr. Hazen's office), and by assistants in Colonel Miller's office.

Contracts.—Plans and specifications for the more important parts of the works were completed in December, 1902, and adopted; and proposals were received under them in January, 1903.

The prices bid in these proposals somewhat exceeded those of the preliminary estimate upon which the appropriation had been based, and all the proposals were rejected. Shortly afterward, Congress appropriated an additional sum, and proposals were again invited. As a result, a contract for about one-half the work, comprising most of the masonry structures, earthwork, grading, etc., was awarded to Cowardin, Bradley, Clay and Company; a contract for the piping, drainage and some other parts, to the Brennan Construction Company; and a contract for the filter sand and gravel to L. E. Smoot.

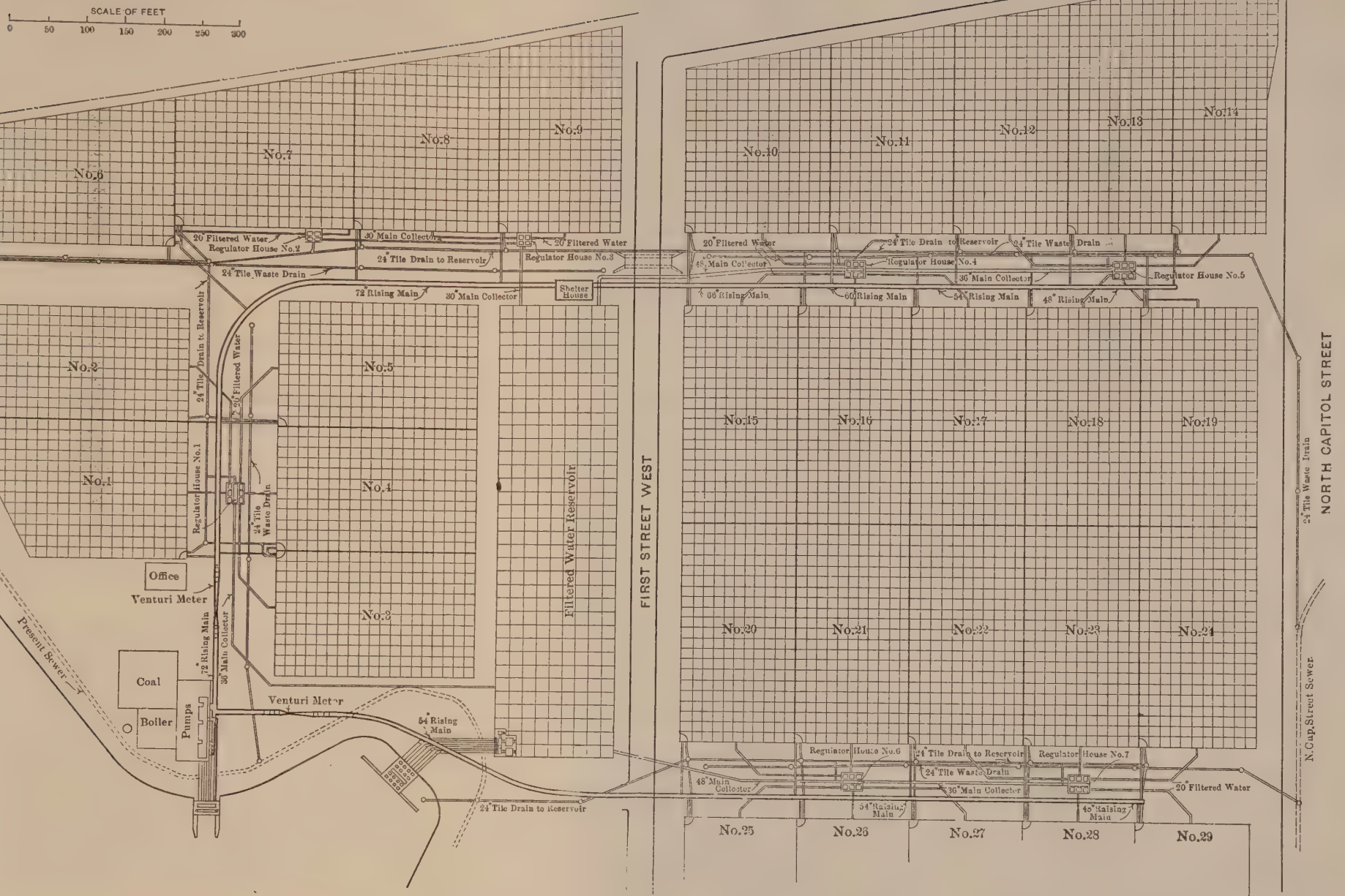
Under subsequent proposals, contracts for pumping machinery were awarded to Henry R. Worthington; for boilers, to Babcock and Wilcox Company; for valves, to the Coffin Valve Company; for Venturi meters, to the Builders' Iron Foundry; for special castings, to the Wilkinson Manufacturing Company; for sand washers, to the Norwood Engineering Company; and for sand bins, to Rudolph S. Blome and Company. Many smaller contracts were let from time to time.

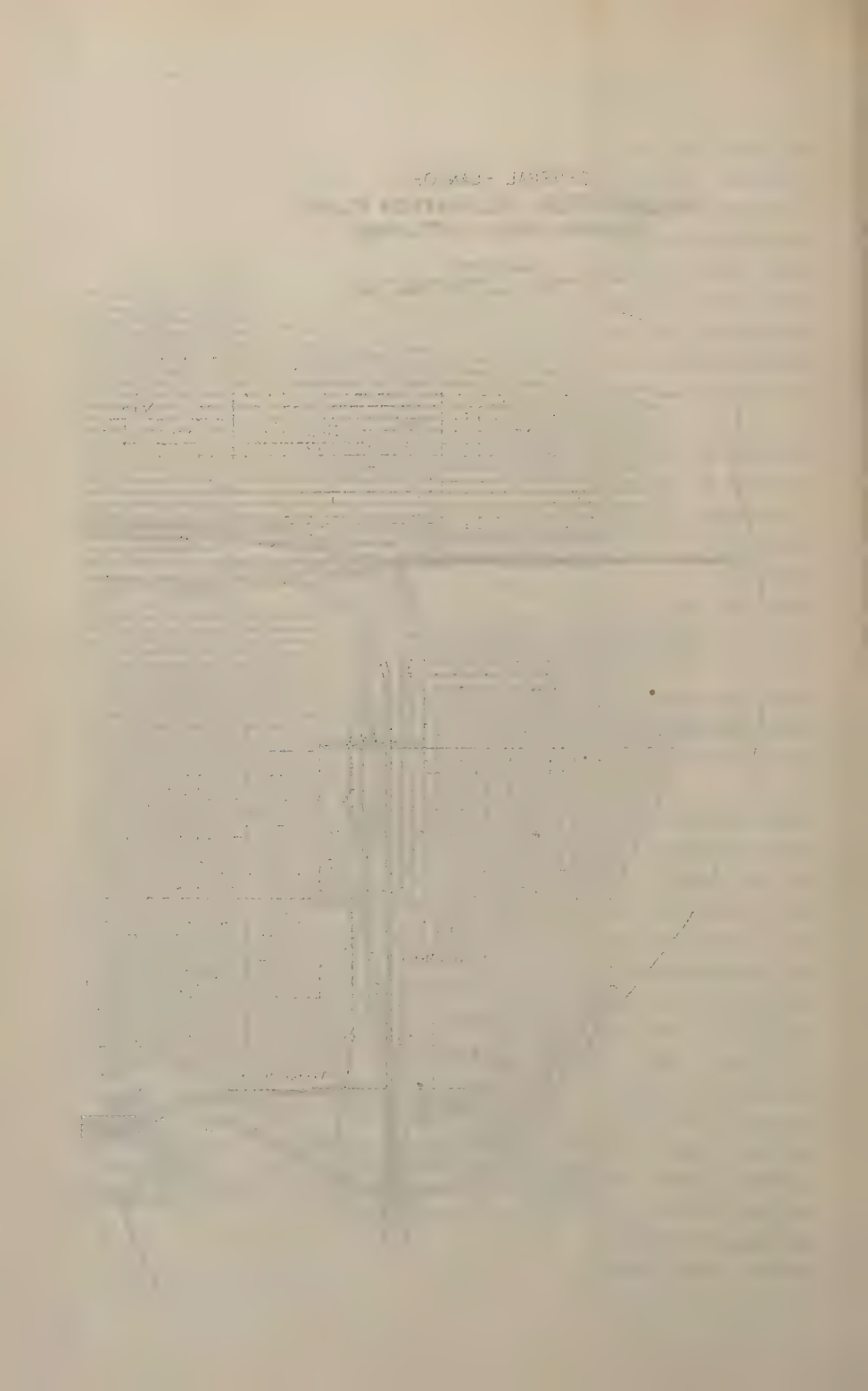
In addition to the work done by contract, considerable was done by day labor. Many of the concrete foundations were built in this way, as well as the regulator-houses, pumping-station, and laboratory and office buildings.

General Plan of the Works.—The filters are immediately adjacent to the Washington City Reservoir, and have a greater elevation. Water is lifted from the reservoir to the filters, the lift ranging from about 20 ft. when the reservoir is full, to 30 ft. or even more when the water is drawn down. Water is pumped direct to the filters, and flows from them through regulator-houses to a pure-water reservoir holding about 14 000 000 gal., and with a flow line 18 ft. above that of the Washington City Reservoir. The waste drainage is in general taken to the sewers of the District of Columbia, but arrangements have been made so that that part of the waste which is of as good or better quality than the raw water can be returned to the Washington City Reservoir, thereby saving it and adding, to that extent, to the net capacity of the aqueduct.

Officers in Charge.—The late Colonel A. M. Miller was in charge of the work from its inauguration until his death, September 18th, 1904. Captain W. P. Wooten was in direct charge of the

GENERAL PLAN OF
WASHINGTON, FILTRATION PLANT.
SHOWING MAIN PIPE LINES.





work under Colonel Miller's direction, and succeeded him temporarily. Colonel Smith S. Leach took charge of the work in November, 1904, Captain Wooten remaining in direct charge of the filter plant. On August 1st, 1905, Lieutenant-Colonel R. L. Hoxie, M. Am. Soc. C. E., succeeded Colonel Leach in charge of the work. Lieutenant E. J. Dent was assigned to the work, and succeeded Captain Wooten in direct charge of the filters on May 10th, 1905. Captain Spencer Cosby, M. Am. Soc. C. E., succeeded Colonel Hoxie in December, 1905, and is now in charge.

One of the writers, Mr. Hardy, as First Assistant Engineer, and afterward as Superintendent of Operation, has been connected with the work from its inauguration to the present time. The other writer, Mr. Hazen, was connected with the preliminary investigation, above mentioned, and with the preparation of plans and other matters connected with it, as Consulting Engineer, for nearly the whole period.

Description of Work.

The construction of the work is in general shown by the plans and photographs which accompany this paper.

Masonry.—All the masonry structures are of concrete, and the general design follows closely the lines of the Albany filters.*

The floors are of inverted, groined arches, carrying piers with a slight batter near the bottom. The walls are of concrete, built in sections not exceeding 30 ft. long, the joints being tongued and grooved. The roof is of elliptical, groined arches, much like those at Albany, but with a deeper cut over the piers, so that the average thickness of the roof is only 0.61 ft., a saving of 14%, as compared with Albany.

The tongued and grooved joints in the walls were made by fastening a plank to the forms for the ends of the wall blocks first built. This formed a groove. The tongue was formed when the second block was built. Generally, the tongues had a slight batter and were at least three times as wide as long, to prevent them from breaking off when movements occurred.

Parapet walls were built over the outside walls along the courts, in order to save space. Generally, it is better to cover all the masonry with embankments, but in this case the site was so limited

* *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 244.

in area that the space which could be saved had a value more than equal to the concrete in the parapet walls, and in the extra thickness of the walls below, which was necessary in connection with it.

Generally, the outside wall acts simply as a beam bearing against the floor blocks and the roof blocks, and holding the earth pressure. The embankments beyond the walls serve to support them and to take the thrust of the vaulting (amounting to 4 000 lb. per lin. ft.), as well as the pressure from the filtering materials and the water inside the filters.

In some cases, owing to the limited space, it was not possible to build embankments thick enough to take these pressures safely, and at these points the thickness of the concrete was increased. It was increased a little where the embankment fell only somewhat below the standard, and much more where the embankment was more deeply cut. In a few cases the concrete was made so thick as to be substantially stable without earth backing, and the embankment was cut until it barely sufficed to cover the masonry.

In places where the tops of the filters are lower than the levels of the adjacent street, the building lines of the filters were put well up to the street lines, so that the embankments required for the street extended somewhat over the tops of the filters. At one place, where the street is much higher than the filters, the fill over the filters was considerably increased; and in this case a number of ventilator manholes were omitted, and the dimensions of the piers and vaulting were increased to carry the additional load safely.

Settlement of Masonry Structures.—An interesting feature of the masonry work was caused by the settlement of parts of the work. The highest part of the site was 65 ft. vertically above the lowest part. This was all graded to allow the filters to be at the same level. The maximum depth of cut was 35 ft., and the maximum height of fill 30 ft. Approximately, 5 acres of masonry structures were built upon fill. The filling was made with material obtained in the excavation, which was generally gravel mixed with clay. The material was excellent, and it would have been difficult to secure anything better for the purpose. It was placed in thin layers, and rolled thoroughly with heavy, grooved, steam rollers. Embankment built in this way was harder, as tested by the distance that an iron bar would penetrate, than the natural undisturbed

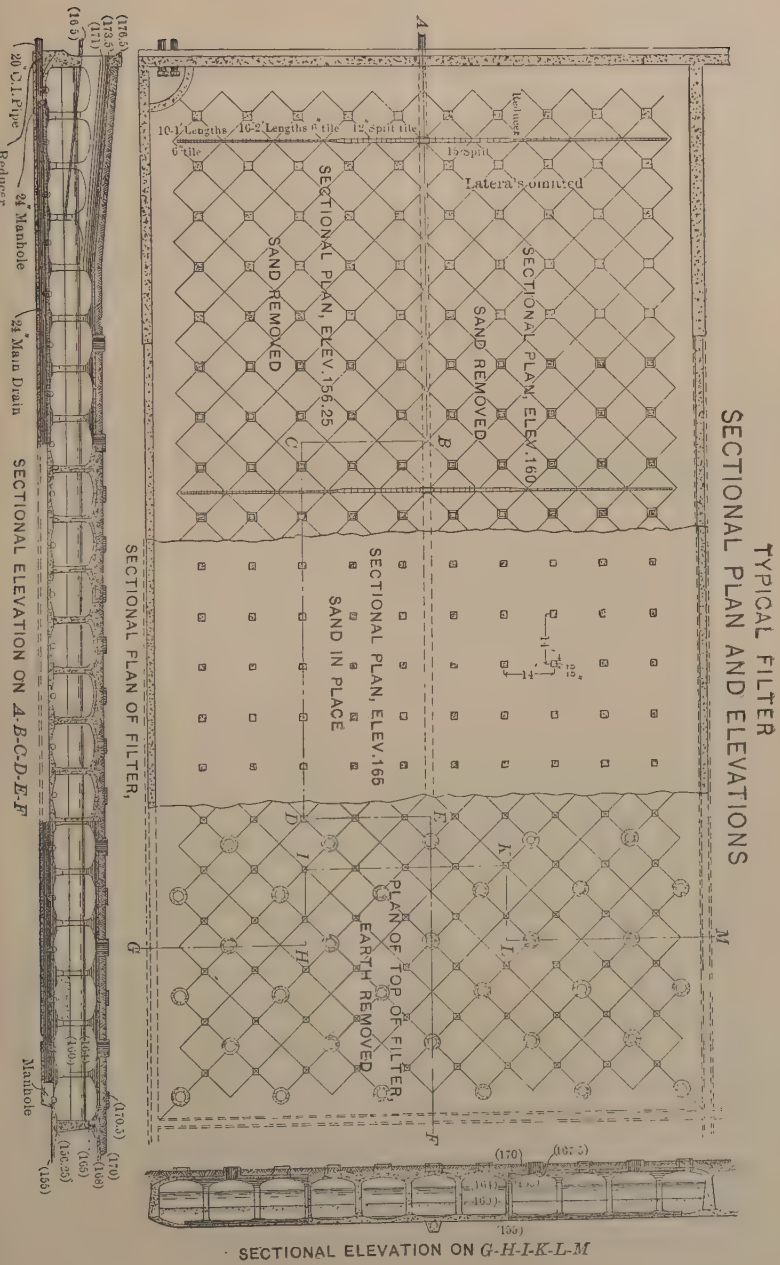

TYPICAL FILTER
SECTIONAL PLAN AND ELEVATIONS
SECTIONAL PLAN, ELEV. 160
SAND REMOVED
SECTIONAL PLAN, ELEV. 156.25
SAND REMOVED
SECTIONAL PLAN, ELEV. 165
SAND IN PLACE
PLAN OF TOP OF FILTER,
EARTH REMOVED
Latera's omitted
10-1' Lengths
16-2' Lengths
12" Split tile
6" tile
15" Split
Reducer
SECTIONAL PLAN OF FILTER,
SECTIONAL ELEVATION ON A-B-C-D-E-F
SECTIONAL ELEVATION ON G-H-I-K-L-M
20" C.I. Pipe
Reducer
24" Manhole
24" Main Drain
Manhole

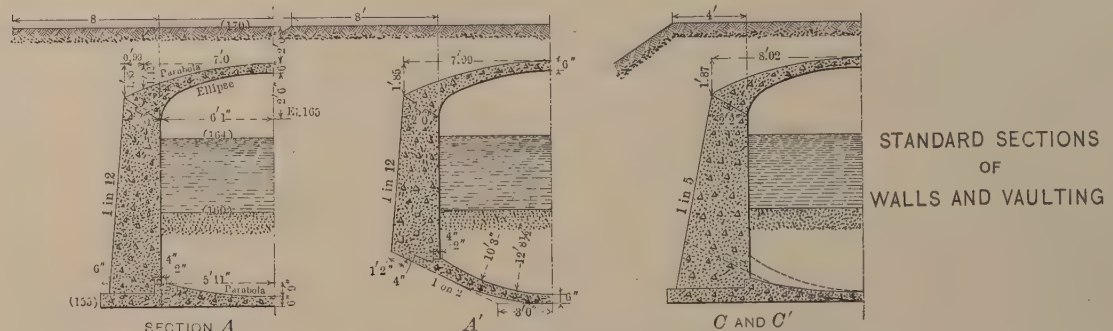
FIG. 1.

material below the frost line, and seemed to be, in every way, as capable of carrying the loads to be placed upon it.

Notwithstanding this, the masonry, wherever it was built upon fill, settled more or less. As a maximum, the settlement reached about 6 in.

The most serious break caused by settlement occurred at the entrance of Filter No. 27. One side of this entrance was on undisturbed material. The natural slope of the ground at this place had been steep. A short distance to one side, the masonry was constructed on a deep deposit of fill, most of which had been made prior to the acquirement of the property by the Government. This material seemed to be solid, but there must have been a soft layer beneath, for a settlement of 6 in. took place in one part of it, and this maximum settlement was within a distance of about 50 ft. of undisturbed material which did not settle at all. The inequality of settlement was too great for the masonry to adapt itself to, and a number of large cracks opened and one section of the vaulting fell. This happened shortly after the filter was filled with water for the first time. The settlement took place rapidly, and, as in other cases, seemed to come to a definite end in a short time. One section of the vaulting and four piers were rebuilt. The replaced section of vaulting was in the inclined entrance, and the general weakness of the structure at this point, as will be mentioned further, was a contributing cause to the unusual damage sustained. In a few other cases cracks in the masonry were produced by this settlement. Generally, however, the movement only caused openings of the tongued and grooved joints of the wall blocks, and slight readjustments of the floor and vaulting blocks. A few of the largest cracks were grouted. Smaller ones were caulked with oakum, which had the advantage of leaving them so that a further movement would produce no damage to the concrete. The openings between the floor and vaulting blocks were seldom large enough to permit of caulking, and nothing was done to them.

The greatest settlements were where the grading, with reference to the use of the land for building purposes, had been done before the property was purchased by the Government. Some of this grading had been done years before. The fills made in this way were in general excavated, in the course of the work, to the original


STANDARD SECTIONS
OF
WALLS AND VAULTING
SECTION A
A'
C AND C'
SECTIONS OF WALLS USED FOR DIFFERENT WIDTHS OF EMBANKMENTS.
DEEP FOUNDATIONS USED IN SHALLOW CUT OR ON FILL. RAISED FOUNDATIONS
(DENOTED BY PRIMED LETTERS) USED IN DEEP CUT.

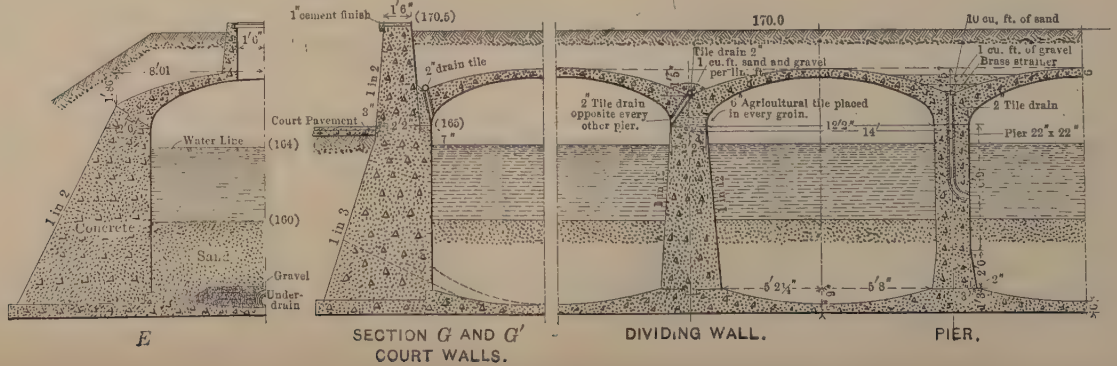

E
SECTION G AND G'
COURT WALLS.
DIVIDING WALL.
PIER.

FIG. 2.

surface, and refilled with embankments made as above described, before the masonry was placed, but in some cases it was difficult to tell just where the old surface had been. In other places where it had been under heavy loading in deep fill, it seemed to be as solid and as little likely to settle as the original material, and such places were left intentionally, as it was believed that they were sufficiently solid to carry the loads to be placed upon them.

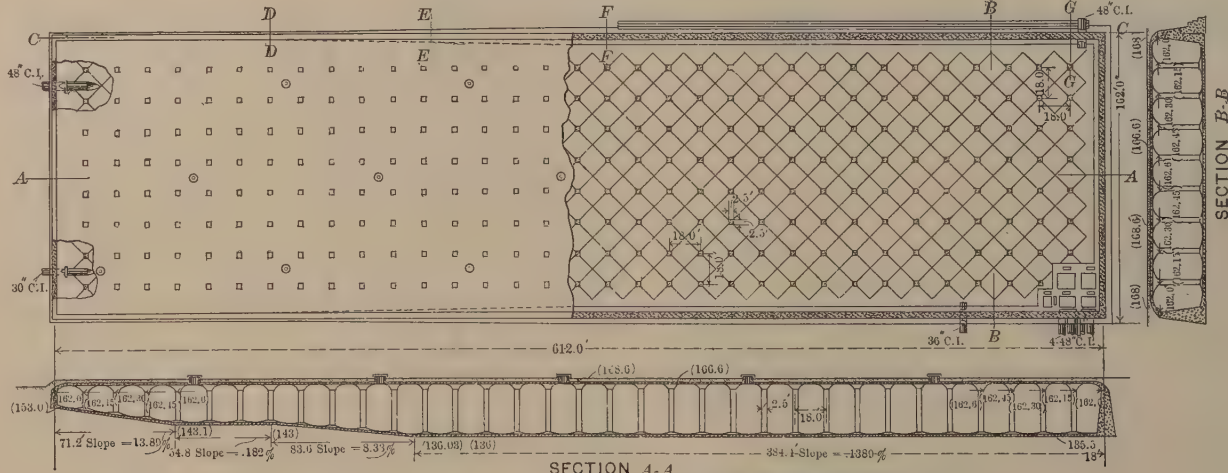
It is difficult, or rather impossible, to determine from the old records just how much of this old filling was left in place. It is known that there was considerable of it, and the greatest settlement seems to have occurred over those places where it was deepest.

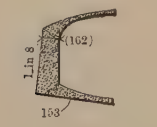
In some places the filters were built 1 or 2 in. higher than the intended grade, so as to approximate more nearly the desired conditions after settlement. The settlement of the masonry does not in any way affect the stability or usefulness of the plant. The greatest practical inconvenience comes from the differences in level of the filters resulting from it.

Cracks in Entrances.—Each filter is provided with an inclined entrance. Such entrances have been used in other covered filters, and are designed to facilitate carrying sand in and out. With the system of sand handling adopted at Washington, they will not ordinarily be used for that purpose. They simply serve as a reserve which can be used in case the sand-handling apparatus fails to perform its work.

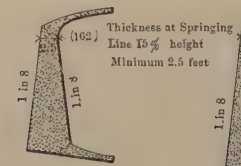
The line of travel in these entrances passes necessarily through the plane of the vaulting, and the vaulting is raised over them. The concrete structures built at this point are larger and heavier than the normal vaulting, and are intended to be strong enough to meet the extra strains which occur at this point. In the Albany filters, where substantially the same design was used for this part of the work, this result was accomplished. In Washington, with a much greater extent of vaulting, there were expansions and contractions of greater magnitude than occurred at Albany, and in some cases settlements of the foundation were also a factor. The entrances were forced to act to some extent as expansion joints, particularly in parts of the work where the concrete was placed late in the fall when the temperature was low. In those places there were movements in the vaulting, with the higher temperature in the follow-

PLAN AND SECTIONS OF FILTERED WATER RESERVOIR.

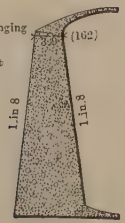




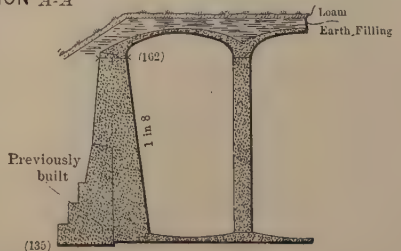
SECTION D-D



SECTION E-E



SECTION F-F



SECTION G-G

FIG. 3.

ing summer, which cracked the concrete at the entrances. The movements were such as to break several piers, and it was necessary to replace them. The cracks for the most part were capable of being grouted and repaired, but it is clear that the design of entrances, in works where the vaulting is uninterrupted and extensive, should be modified.

Diagonal Vaulting.—At several places the shape of the site was such that it was necessary to place the outside walls on lines not parallel with the pier lines. This involved complications in the construction of the floors and vaulting. The plan adopted was to place the outside wall on a diagonal line and to carry a cylindrical arch of the usual shape springing from it for a distance not exceeding one-half the usual span of the arch. The piers were built on the usual plan, as near the wall as possible, and the groined-arch vaulting was carried up from them. The customary curves were used in all cases, and the various surfaces intersected formed irregular arches which tended to be narrow and pointed like Norman arches. This became well developed at places where the piers were near the walls. The angles of the outside walls were arranged so that the relations of the piers to the walls became the same at regular intervals, and this allowed the special centering to be used over again. In some cases there were small triangular places at the tops between the different surfaces, which were not large enough to warrant the construction of another pier. These were built perfectly flat, a small quantity of steel being used in the concrete.

The piers are all at 14-ft. centers, except that the span was made 4 in. shorter for five of the filters, in order to fit the dimensions of that part of the site with a fairly regular design.

In the pure-water reservoir the piers are 18 ft. apart on centers. This greater span was adopted to avoid the use of piers too high for their width. The height of these piers is about 25 ft., or about ten times their width.

The floor of the pure-water reservoir is not level. Part of the site was naturally at, or had been excavated to, a level below that at which water could ordinarily be used from it. It was cheaper to carry the floor of the reservoirs at the level of this material than to fill it and build a shallower reservoir, and in emergencies the water stored in this lower part of the reservoir would be of value.

It was not worth while excavating the remainder of the site to this depth, and consequently a sloping floor was used, thereby largely reducing the quantities of excavation, and of masonry in the outside walls, without materially reducing the effective storage capac-

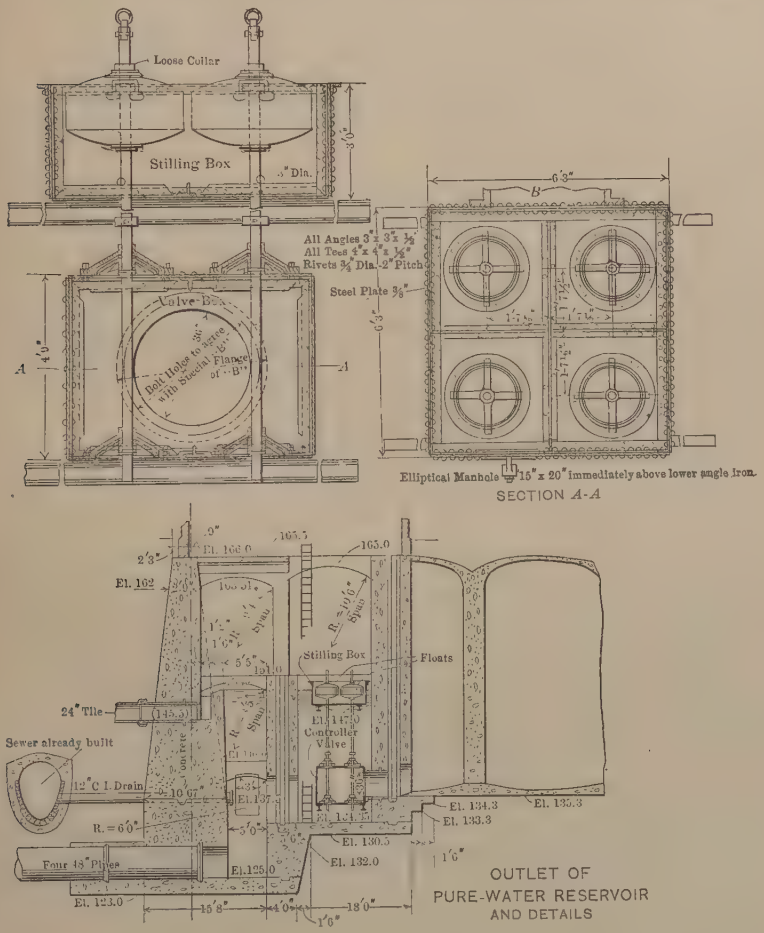


FIG. 4.

ity. The sloping floor was built in the usual way, by simply raising the grades and setting the forms on a slope, strips being nailed to the tops on the lower side to level up for the piers.

The outside walls of the pure-water reservoir are like the filter walls, but much heavier on account of their greater height and the

greater thrust of the arches. The section was varied according to the height of the wall in such a way that the thickness of the springing line was 15% of the height of the wall, with a minimum thickness of 30 in., while the thickness at the bottom was 40% of the height of the wall. A lighter steel-concrete wall, or a buttressed concrete wall, might have been used, with some saving in expense.

Drainage System.—The drainage system of the Washington filters was designed in general upon the same principles as those followed in the Albany filters.

Owing to the increased size of the filters it was necessary to have lateral drains of greater capacity, and split 12-in. pipes were used in those places where round 6-in. pipes were not sufficient. Tests of 12-in. pipes showed that they were amply strong for the service. The pipes were not perforated, as at Philadelphia, because the open joints have an area far greater than the area of the drains, and furnish all needed space for the entrance of water.

Central Drains.—The central drains, as at Albany, are below the floors of the filters, and were built first. This permitted the inverted arches of the floors to pass over them without interruption of any kind. It would have been more convenient to have the central drain higher, but this would have necessitated cutting the inverted arches, which would have involved a tendency to movement by the floor blocks and all the masonry.

In one respect the design of the central drain was different from those in filters formerly built. This peculiarity had been incorporated previously in a design for filters at Pittsburg, made by one of the writers, but that design was not carried out there. It consists of the use of compensating orifices, to balance the frictional resistances and velocity heads in the main drains. When a filter has just been cleaned and is put in service, the frictional resistance of the sand is very small, usually only 1 or 2 in. The frictional resistance and velocity head of the water flowing in the drain of a filter must be kept within very low limits, as otherwise their effect would be to cause that part of the filter nearest the outlet to operate at a rate much higher than intended; while the remote parts of a filter would hardly filter at all until the nearer parts were to some extent stopped by the clogging of the sand. To avoid this condition, velocities in the main drain as low as 0.6 ft.

per sec., or less, are necessary, and this results in a very large central drain for filters as large as those at Washington. The total loss of head in these drains is not serious, from the standpoint of the level of the water delivered; but its disturbing effect upon the equality of the rate of filtration is a most important matter.

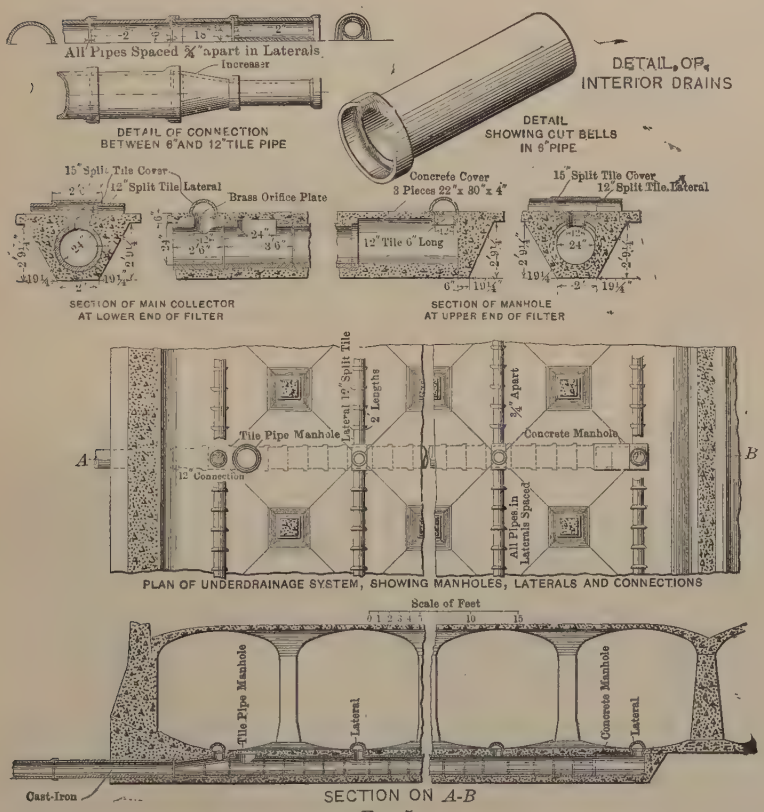


FIG. 5.

The principle of the compensating orifice is to introduce resistances, at each point where laterals enter the main drain, equal to the combined frictional resistance and velocity head in the main drain from the most remote point to the point in question. If the computations are correctly made, this would result in making the resistance from any point back of a compensating orifice, through

it, and thence to the outlet, precisely the same as from any and all other similar points. The result should be to draw water at exactly the same rate of filtration from each section of the filter. As all the losses in the underdrains and compensating orifices change with the changes in rate in nearly the same ratio, namely, as the square of the rate of filtration, it follows that a compensation adjusted for one rate of filtration is substantially right for any other rate. The fact that the loss of head in the sand varies directly as the rate, instead of as the square of the rate, does not enter into the comparison; for, at a given rate, this loss is the same for all parts of a filter, and therefore there is no tendency from it to produce unequal rates of filtration.

The compensating orifices were brass discs, with circular openings cut to the computed diameters and placed in the entrances to the main drains, all other parts of which were finished water-tight, or substantially so. By this construction the main drains were 24-in. tile pipe throughout, which is sufficient; but without the orifices it would have been necessary to use 36-in. pipe. The saving effected by the use of the orifices was roughly $500 per acre, or $14 500 for the plant.

Gravel.—The total depth of the gravel layer midway between the piers is 12 in., decreasing to 3 in. at the piers. Three grades of gravel were used. The material throughout was crushed stone, either trap or granite, limestone and material containing lime in large quantity being excluded. The lower layer of coarse gravel was 7 in. thick, coming just over the tops of the drains. The two finer layers were each $2\frac{1}{2}$ in. thick. The gravel was kept 2 ft. away from the outside walls. The sizes and methods of placing were practically identical with those used at Albany.

Sand.—The sand is, in a way, the most important part of the filters. To secure the best sand that can be reasonably obtained is important. The specifications for sand were much like those used at Albany, but required a somewhat greater freedom from fine particles, as experience had indicated that this was necessary to secure the best results in the filtration of river waters containing as much clayey turbidity as the Potomac water. It was expected when the specifications were drawn that the sand dredged from the Potomac would be used. The specifications were general, and ad-

FIG. 1.—INTERIOR VIEW OF FINISHED MASONRY, SHOWING ARRANGEMENT OF LATERAL DRAINS AND GRAVEL LAYERS.

FIG 2.—SAND PIPING IN A FILTER BEFORE PLACING FILTERING MATERIALS.

mitted any sand of the required quality without regard to its source. The contractor actually furnished sand from a bank at Laurel, Md., on the main line of the Baltimore and Ohio Railroad, half way to Baltimore. This bank was probably of Tertiary origin, and consisted of layers of clay and sand. The sand in the sand layers was of good quality, except that more or less clay was distributed through it. The layers of clay ranged in thickness from a few inches to several feet, and the mixing was such that it was not possible to take the sand without the clay.

The method of securing and preparing filter sand of the requisite cleanliness and of the quality specified was interesting and novel. The procedure was as follows: The sand was excavated from the bank with steam-shovels, taking the mixed material, to a depth often reaching 20 ft. The material obtained in this way consisted mostly of sand, but large and small lumps of clay were always mixed with it, and the top soil was not separated. The proportion of the material which could not form part of the filter sand was rather large. The sand was loaded on cars, which carried it on temporary tracks to the screening and washing plant built close to the main line of the Baltimore and Ohio Railroad.

The material was first dumped from the cars through a coarse grating which separated many of the largest lumps of clay. It then passed through a revolving screen, with holes about 2 in. in diameter, which removed further quantities of clay in lumps. It was then taken by a link-belt elevator to the top of a timber trestle, and discharged into a revolving screen, with round holes having a size of separation of about 4 mm. Water jets played upon this screen and facilitated the passage of sand through it, while much fine gravel and some additional lumps of clay were removed. The specifications provided that the sand must be free from particles more than 5 mm. in diameter, and the screen secured this result. The material passing through the screen consisted of the sand, together with a large quantity of clay, partly pulverized and partly in lumps, all carried by a considerable quantity of water. The mixture then passed to a series of pug-mills. The revolving arms in these broke up and pulverized the remaining clay lumps. This treatment was necessary for a material containing clay in lumps, but would be unnecessary for sand not containing such material.

The pug-mills incidentally served to separate a portion of the clay from the sand, for an excess of water entered them, and extremely dirty water was constantly wasting over their tops, while the sand was drawn out from points near the bottoms in much the same way as it was subsequently drawn from the sand washers.

The mixture of sand, clay and water leaving the pug-mills next passed to the washers. These washers consisted of three long narrow boxes with bottoms having slopes of 1 in 6 to the point of discharge. The boxes were 16 ft. long, 24 in. wide, and 18 in. deep at the upper end. There were four pipes, perforated for their entire length, in the bottom of each box, the holes opening directly downward. Water was forced through these pipes at a rate of about 1 cu. ft. per min. per sq. ft. of box area. This water went upward and overflowed into a trough running lengthwise of the box at the top. The mixed materials entered this box at the upper end, flowed through it, and were discharged at the lower end from the bottom. There were, therefore, two movements in each box: first, a movement of wash-water upward from the bottom of the box to the top and out through the waste overflow; and second, a forward movement of sand from one end of the box to the other. The upward movement of water, starting from the whole area of the bottom and overflowing from most of the area of the top, kept the sand in a semi-suspended state and practically in the condition of quicksand.

Under these conditions the larger particles of sand rapidly sank to the bottom while the finer particles were carried to the top. The sand at the bottom was in contact with the clean water as it first entered the box, while, by controlling the quantities of sand let in and drawn out, the finer particles could be forced to the top and out through the waste overflow to any desired extent. The level of the sand in the box was usually carried not more than about 6 in. below the surface of the water.

As the sand in the box was in the state of quicksand, it was possible to draw it out, through a gate placed just above the bottom at the lower end of the washer, in the form of a fluid containing very little water. Generally, 10 parts of the mixture drawn from the outlet contained 9 parts of solid sand. The mixture fell into a large hopper, from which a gate allowed it to flow from time to time into cars on a side-track below, often without further separation of

SKETCH OF
SAND WASHER BOX
USED BY CONTRACTOR
AT WASHINGTON, 1904.

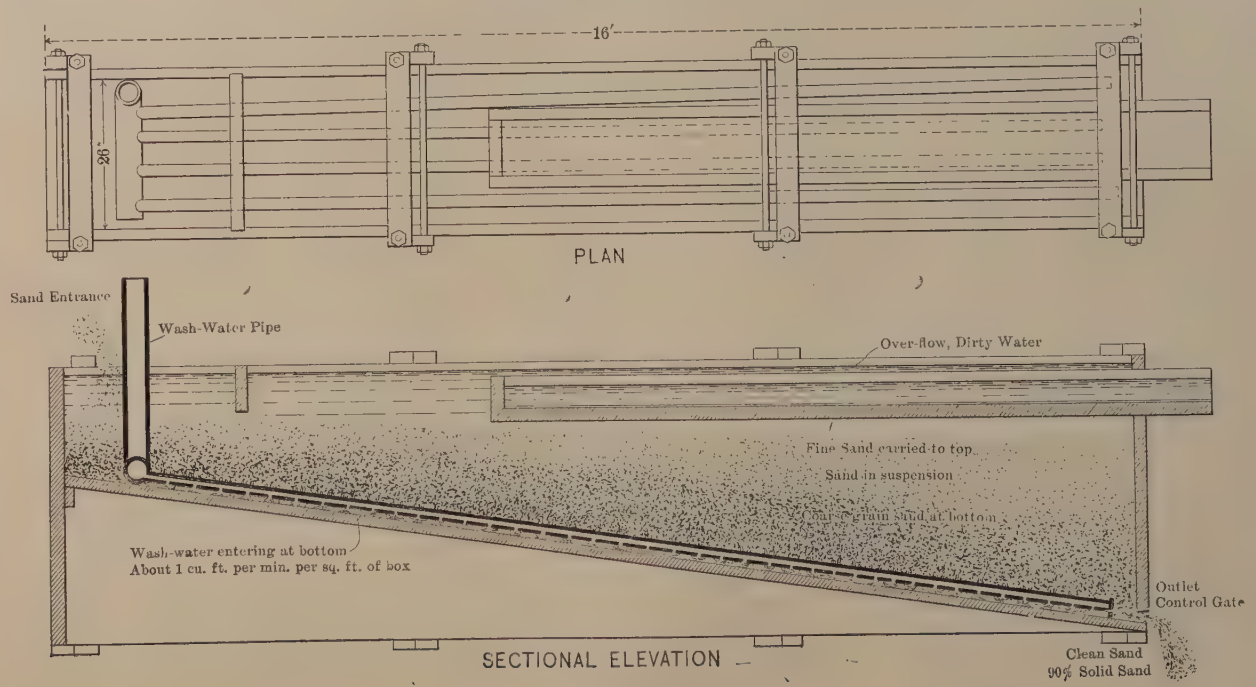


FIG. 6.

water, except as it gradually drained out through the cracks in the hopper and in the bottoms of the cars.

In general, it was found that 1 cu. yd. of sand per hour could be washed for each square foot of box area, and sometimes a larger quantity was passed.

A washing box of this character was first designed by one of the writers for use in preparing filter sand at Yonkers, N. Y.* The same type of box was used in preparing all the sand placed in the filters at Providence, R. I., and has also been used elsewhere.

The separation of the clay from the sand in such large quantities and so cheaply was an achievement which would hardly have been regarded as possible at the time the contract for filter sand was made, and the use of this process cheapened the sand washing very greatly, the actual cost to the contractor being far below the contract price.

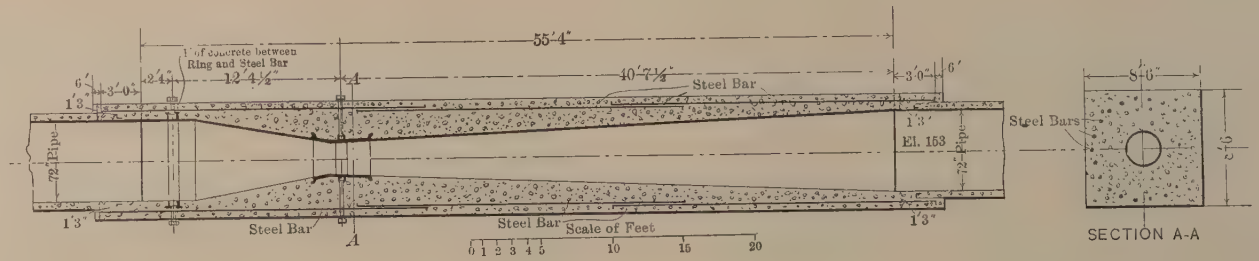
Although exact figures are not at hand, it appears that the volume of water used in washing the sand was not more than five or six times that of the sand. The wash-water was obtained from a small creek nearby, and was pumped through a 10-in. pipe. After rains the water in this creek was quite turbid, but this turbidity did not interfere materially with the washing, or with the quality of the sand produced.

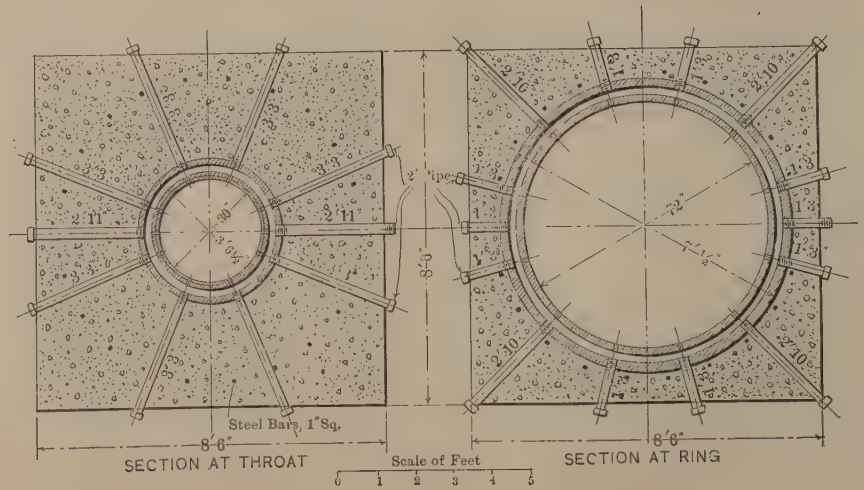
In a working day of 10 hours, more than 900 cu. yd. of filter sand were frequently produced, and, had it been possible to handle the sand at the filters more rapidly, the plant could have worked at night, with a greatly increased output.

The specifications provided that the filtering sand should be entirely free from clay. This specification had proved sufficient in securing sand from river deposits and from sand banks of glacial origin. It did not prove satisfactory in the case of this sand, as the raw material contained large quantities of clay. The clay stuck to the particles of sand on drying, and the ordinary mechanical analysis, by sifting the material in a dry state, was inadequate to show its presence or amount.

It becomes apparent at once that a method of measuring the amount of clay in the sand must be found and used, and definite

* Described in *Engineering News*, July 14th, 1904, and in *The Engineering Record* July 2d, 1904.





72-INCH
REINFORCED CONCRETE
VENTURI METER

FIG. 7.

limits set to the amount of clay that could be present, which should be substantially equivalent to the requirements of the specifications.

The method adopted of determining the amount of clay was as follows: A weighed quantity of sand, usually 25 g.—but less if there was considerable clay in it, and more if there was but little—was agitated for some minutes with several times its volume of water. The sand for this purpose was taken directly from the washers, and was not dried, as drying increased the difficulty of getting the clay into suspension. If the sand had dried before testing, it was necessary to keep it moist and agitate it for some time to get all the clay loose. When this was accomplished the mixture was made up to a volume of 1 liter in a graduated glass. This was allowed to stand for 1 min. The turbidity of the supernatant fluid was then taken by observing the depth below the surface that a platinum wire could be seen, by the method of the United States Geological Survey.

These observations were taken in the graduated glass for convenience. This was not strictly in accordance with the official instructions, but it was more convenient, and the comparative results were good. Jackson's turbidimeter was used with good results for night work, but the rod was preferred by the inspectors when it could be used. The turbidity of the water thus found was multiplied by the ratio of the volume of the mixture to the weight of sand taken. That is to say, for the quantities above stated it was multiplied by 40. The figures thus represent approximately the turbidity in the sand in parts per million by weight. One part of clay by weight actually produces about two parts of turbidity, because the particles of clay are much finer than the particles of standard turbidity, but this matter is overlooked, and the results are expressed as standard turbidity in parts per million. To get the actual weight of the clay, therefore, the figures should be divided by two.

It was decided after study that a reasonable interpretation of the specification, expressed in terms of turbidity, was represented by 4 000 parts per million, and this limit was rigidly insisted upon. Generally, the sand contained less than 3 000 and frequently less than 2 000 turbidity, the last figure corresponding to less than 0.1% of actual clay by weight in the sand as delivered. That this result

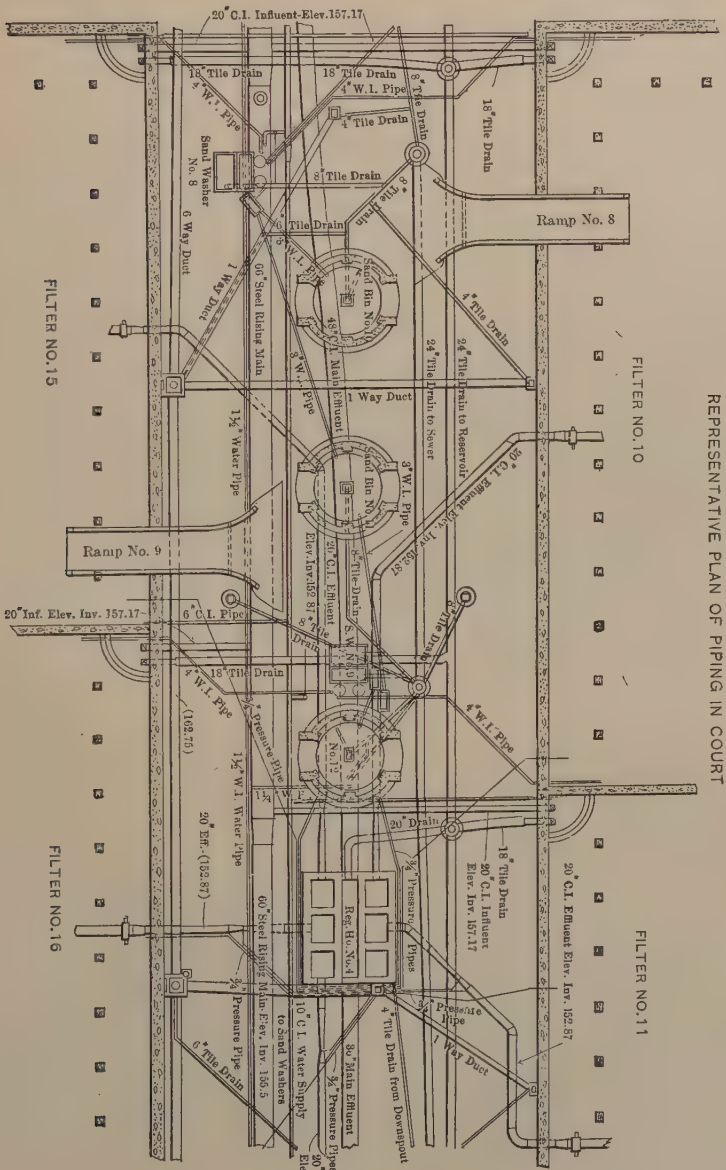

REPRESENTATIVE PLAN OF PIPING IN COURT
FILTER NO. 15
FILTER NO. 16
FILTER NO. 10
FILTER NO. 11
Ramp No. 8
Ramp No. 9
20" C.I. Influent-Elev. 157.17
18" Tile Drain
4" W.I. Pipe
8" Tile Drain
6" Tile Drain
Sand Washer No. 8
6 Way Duct
1 Way Duct
66" Steel Rising Main
48" C.I. Main Effluent
3" W.I. Pipe
24" Tile Drain to Sewer
24" Tile Drain to Reservoir
1½" Water Pipe
20" C.I. Effluent Elev. Inv. 152.87
20" Inf. Elev. Inv. 157.17
6" C.I. Pipe
18" Tile Drain
4" W.I. Pipe
(162.75)
¾" Pressure Pipe
1½" W.I. Water Pipe
20" Eff. (152.87)
20" Drain
18" Tile Drain
20" C.I. Influent Elev. Inv. 157.17
20" C.I. Effluent Elev. Inv. 152.87
Reg. Ho. No. 4
60" Steel Rising Main Elev. Inv. 155.5
10" C.I. Water Supply to Sand Washers
30" Main Effluent
4" Tile Drain from Downspout
1 Way Duct
6" Tile Drain

FIG. 8.

could be regularly secured from a bank where a considerable percentage of the total material was clay is, the writers think, a very remarkable result, indicating both an excellent apparatus and most efficient management, on the part of the contractor, and by the sand inspectors.

Part of the sand-washing plant was duplicated. This was done before the full capacity of the part first built was realized. It was intended to insure against delay in case of accident and to allow an increased output, but the first part did so well that the second part was used hardly enough to test it.

The sand was taken in cars to an elevated siding near the filters, and dumped into hoppers. These hoppers were provided with sand-gates, and carts were driven underneath and loaded from them. These carts were taken over the roofs of the filters, and the sand was dumped through the manholes. Chutes were arranged under the manholes, upon which the sand fell. This broke the force of the fall which, otherwise, might have compacted the sand to an undesirable extent, and also threw it to a considerable distance horizontally. The chutes were revolved, and in this way most of the filter sand was placed directly where it was wanted without further handling. It was necessary to place only a small part of it with shovels.

This method of placing the sand in the filters is so simple and cheap that it has been adopted for regular use in replacing the washed sand in the filters.

The sand settled, on an average, about 5% when it was wet and the filters were placed in service. The average depth of the sand in the filters after settling was 38 in., but different filters were filled to different depths, so that when sand is replaced from the washers in the filters it will go first to the filters having initially the least sand, and a regular regime is thus established from the start.

Piping.—All the piping for taking the filtered water to the pure-water reservoir is of cast iron, 48 in. in diameter, or less. The main pipes for carrying the raw water from the pumps to the filters are of riveted steel. Two lines were provided, one 72 in. in diameter, supplying 19 filters; and one 54 in. in diameter, supplying 10 filters. These pipes are coated with asphalt and entirely surrounded by concrete.

FIG. 1.—MOVABLE EJECTOR AT WORK MOVING SAND TEMPORARILY STORED ON TOP OF A FILTER.

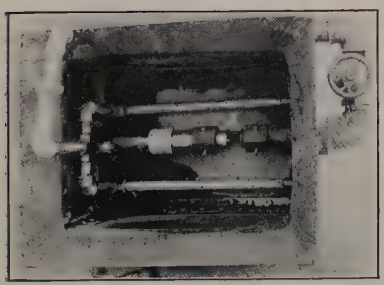


FIG. 2.—MOVABLE SAND WASHER.

FIG. 3.—MOVABLE SAND WASHER.

Venturi Meters.—In each of these lines is built a reinforced concrete Venturi meter. These meters were built with the proportions described by Clemens Herschel, M. Am. Soc. C. E.* One casting was set for the throat of each meter, and another for the entrance. These castings had the usual openings and annular spaces, and were connected with small pipes leading to the registering apparatus in the pumping station. The fittings and registering devices were furnished by the Builders Iron Foundry. The reinforced concrete meters were much cheaper than if made of cast iron, and are perfectly satisfactory.

It would have been possible to build the entire raw-water lines of reinforced concrete, had it not been for the fear of unequal settlements, shrinkage and cracks, and the consequent troublesome leakage of water. The water in these lines is under such pressure that it would usually rise a little above the surface of the courts, were there to be such cracks.

Regulation.—The water is pumped from the Washington City Reservoir directly to the filters, and there are no sedimentation basins at the filters. They are unnecessary because the water has been most thoroughly settled before it reaches the plant; and further, there is no space available for them on the site selected, all of which is required for the filters. The piping from the pumps to the filters is so ample that, at the rate of filtration to be used, the water level upon all the filters, with the gates wide open, is nearly the same. The difference of a few inches between the nearest and most remote filters is nearly constant, and is not troublesome.

It is necessary to regulate the speed of the pumps to maintain the desired water level upon the filters. As close regulation of the pumps is difficult, the regulating apparatus of the filters is made so that fluctuations in the levels of the water on the filters through a range of 6 in. are permissible. On an average of 25 filters in service, 6 in. represents a storage capacity of 4 000 000 gal., which gives ample leeway for the convenient and economical operation of the pumps.

The regulation of the quantity of the effluent from each filter is secured by a gate with a large wheel and a graduated standard upon the filter outlet. The discharge is through a Venturi meter,

* *Transactions*, Am. Soc. C. E., Vol. XVII, p. 228.

with suitable apparatus placed directly behind the controlling gate to show the rate of filtration and loss of head. Watchmen observe and record these data at stated intervals, and change the outlet gate when necessary in order to maintain the desired conditions.

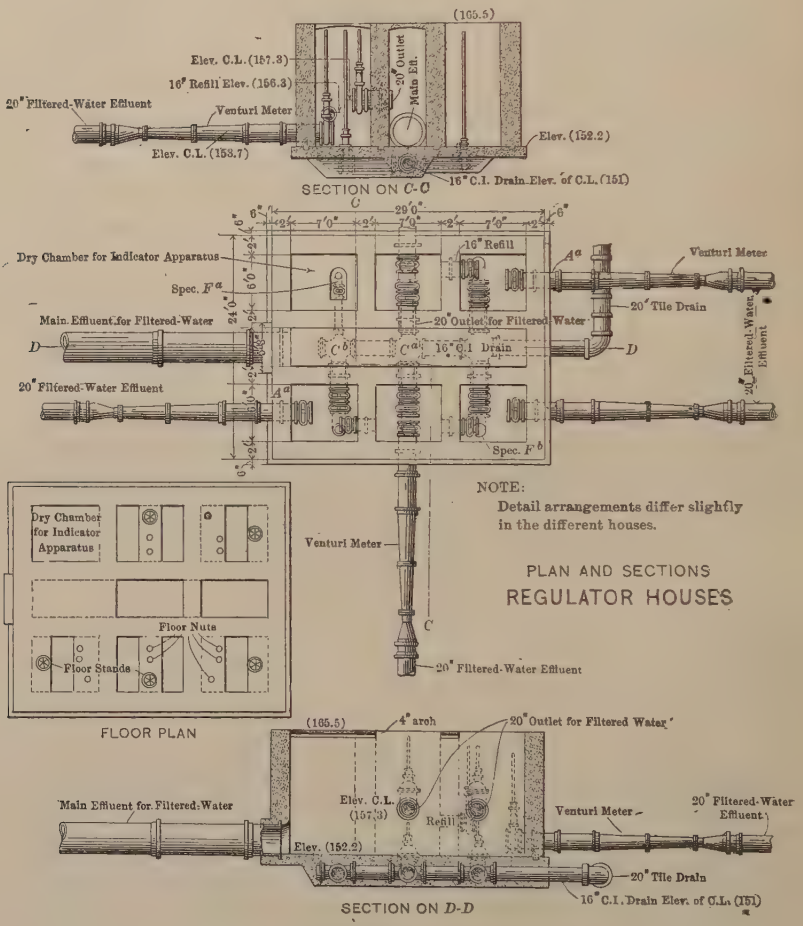


FIG. 9.

This system of regulation was selected in preference to an entirely automatic system, as the required apparatus is much simpler, and no additional labor is involved. This is because watchmen must make the rounds of the filters in any event to see that all is going

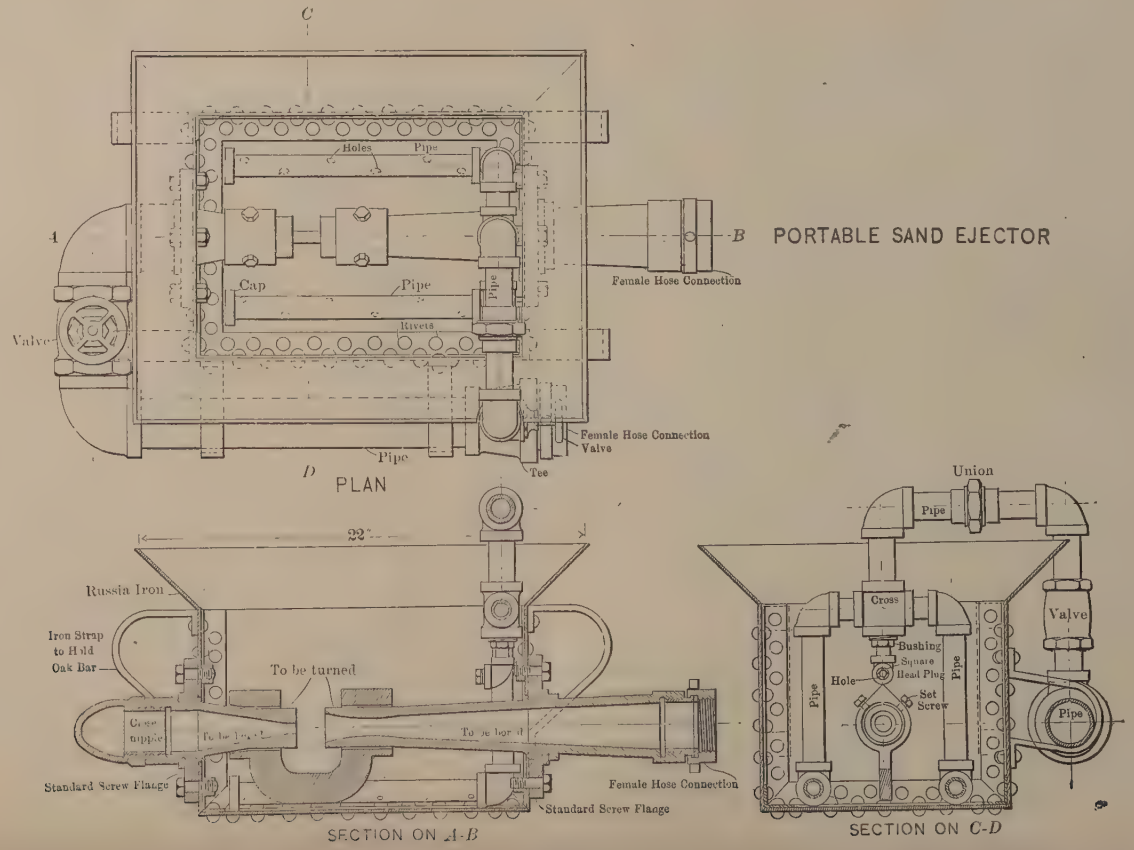

C
Holes
Pipe
A
B
PORTABLE SAND EJECTOR
Cap
Pipe
Pipe
Female Hose Connection
Rivets
Valve
Female Hose Connection
Valve
Pipe
D
Tee
PLAN
22"
Russia Iron
Iron Strap to Hold Oak Bar
To be turned
Grease nipple
Standard Screw Flange
Female Hose Connection
Standard Screw Flange
SECTION ON A-B
Union
Pipe
Cross
Valve
Bushing
Square Head Plug
Hole
Set Screw
Pipe
Pipe
Pipe
SECTION ON C-D

FIG. 10.

well, and it is never necessary to change the adjustments oftener than such rounds would be made.

The pure-water reservoir is arranged so that when it becomes full it will back up water against the filters and reduce the rate of filtration automatically, but the outlets from the filters to the pure-water reservoir are placed at such an elevation that a considerable drop in the pure-water reservoir does not put a corresponding suction or pull upon the filters, but, instead, the loss of head is limited to about 4 ft. Any fall in the water in the pure-water reservoir below the level corresponding to this loss of head has no effect upon the rate of filtration.

The pure-water reservoir has its flow line 18 ft. above the nominal level in the Washington City Reservoir, and 12 ft. above the highest elevation for which provision has been made in the gate-houses of that reservoir.

As the pressure obtained by gravity in the lower part of the city from the works before the filters were built was not large, it would have been desirable to increase it by taking advantage of the higher elevation of the pure-water reservoir. To do this, however, would have required the entire reconstruction of the two gate-houses already in use. These gate-houses were quite elaborate, with provision for all possible by-passing of all parts of the work, and represented a large investment. The designs were such that they could not be easily adapted for handling water at a higher level. To use the water at the higher elevation, therefore, practically meant the abandonment of these gate-houses.

To avoid doing this, the water on leaving the pure-water reservoir passes through controllers which reduce its level to the highest elevation permissible in the old gate-houses. These controllers consist of a series of eight pairs of balanced valves, arranged in batteries of four pairs each. Each pair of valves is operated by a copper float resting on the water in a stilling-box in the discharge chamber. This has the effect of holding the water in this chamber at the desired level.

The balanced valves do not close tightly, but have annular spaces, so that a considerable quantity of water passes them in any event. These spaces were made to pass less water than the assumed minimum rate of consumption, so that the valves are capable of con-

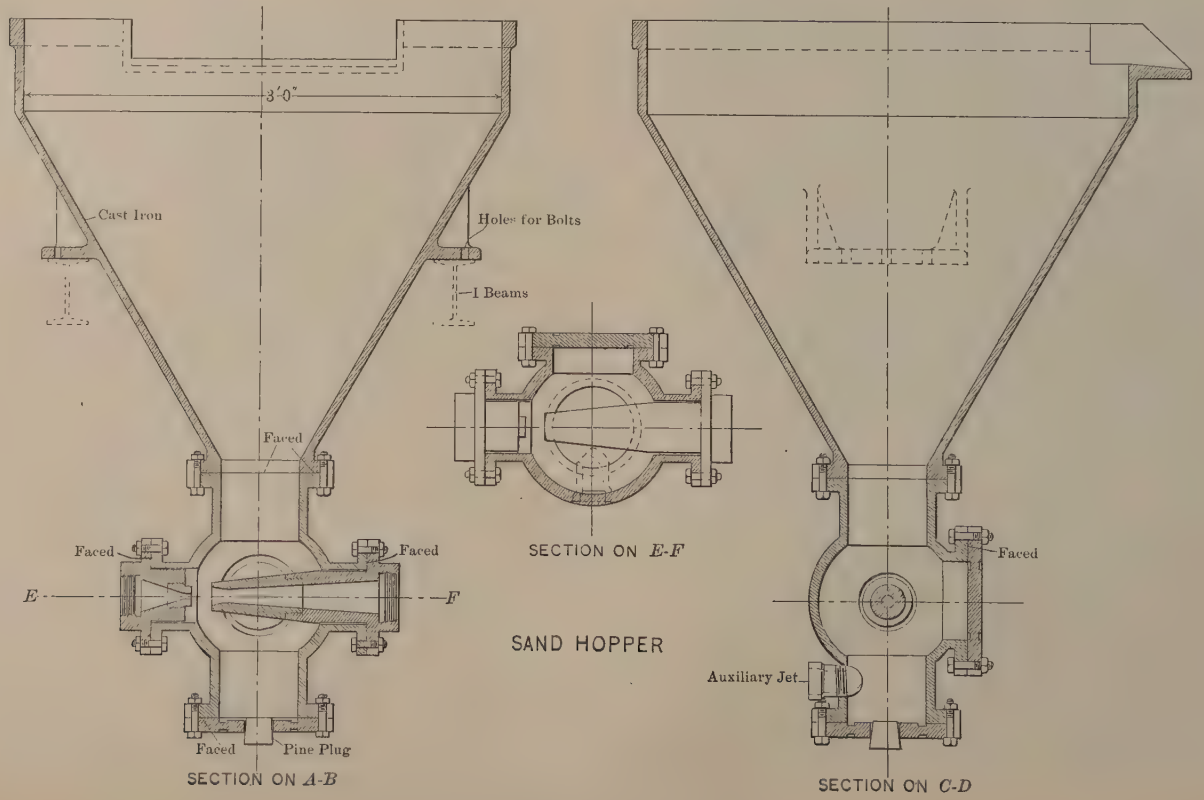

3'0"
Cast Iron
Holes for Bolts
I Beams
Faced
Faced
Faced
E
F
Faced
Pine Plug
SECTION ON A-B
SECTION ON E-F
SAND HOPPER
Faced
Auxiliary Jet
SECTION ON C-D

FIG. 11.

trolling through the entire assumed range in consumption. Both the pure-water reservoir and the discharge chamber are provided with overflows back to the Washington City Reservoir, to prevent the water levels from going above the desired heights. A by-pass which can be operated by hand will maintain the supply when the balanced valves are out of service for any reason.

Provision was made to take a large pipe from the pure-water reservoir directly to the city at some time in the future. An opening through the wall of the reservoir was built and closed with a thin wall easily removed, so that this connection can be made whenever the advantage of the increased head becomes important enough to justify the construction of a pipe line connecting the reservoir directly with the distribution system in the city.

In the meantime, the extra head is lost, but the system has the advantage of furnishing an unusually steady pressure, for the pipes are protected from even the fluctuations in level in the pure-water reservoir.

Sand Washing.—The sand-washing and handling plant is one of the most important features, because the work to be done by it is by far the most expensive part of the operation, and makes up a large part of the total cost of purification. The general arrangements of these appliances are as follows:

The dirty sand is scraped off and piled up by men with shovels. It is then thrown into movable ejectors, which force it by water pressure through lines of hose and pipe to stationary sand washers in the courts. These washers clean the sand and also elevate it to cylindrical storage tanks. These tanks are high enough so that carts drive under them and are filled by opening gates in their bottoms. The carts in turn take the sand over the top of the filters and drop it through manholes upon revolving chutes, in the manner described in connection with the placing of the sand by the contractor during construction.

Sand Ejectors.—The movable ejectors have two novel points in their design. First, the water for bringing the sand into suspension is introduced gently at the bottom of the ejectors and slowly rises as the sand is shoveled into it. It was found by experiments, made under the authority of Colonel Miller, that sand could be more effectively loosened and suspended in this way than by playing water

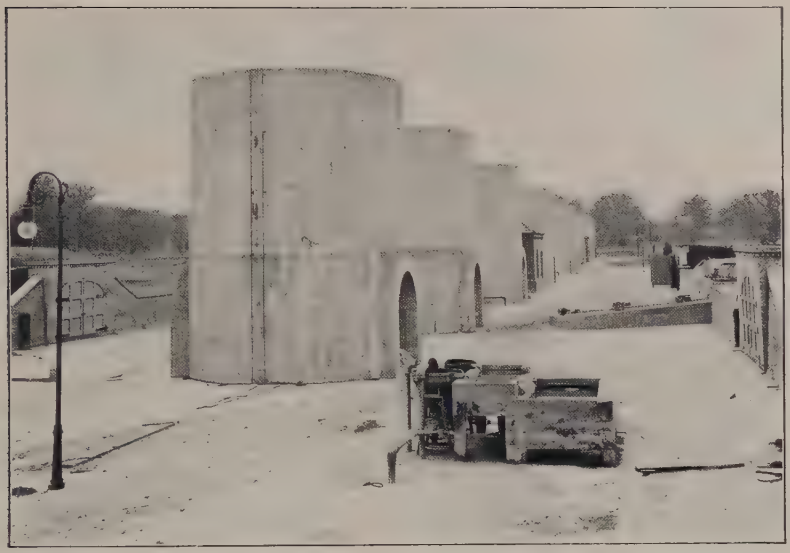


FIG. 1.—GENERAL VIEW OF COURT, SHOWING SAND WASHER, SAND STORAGE BINS, ETC.

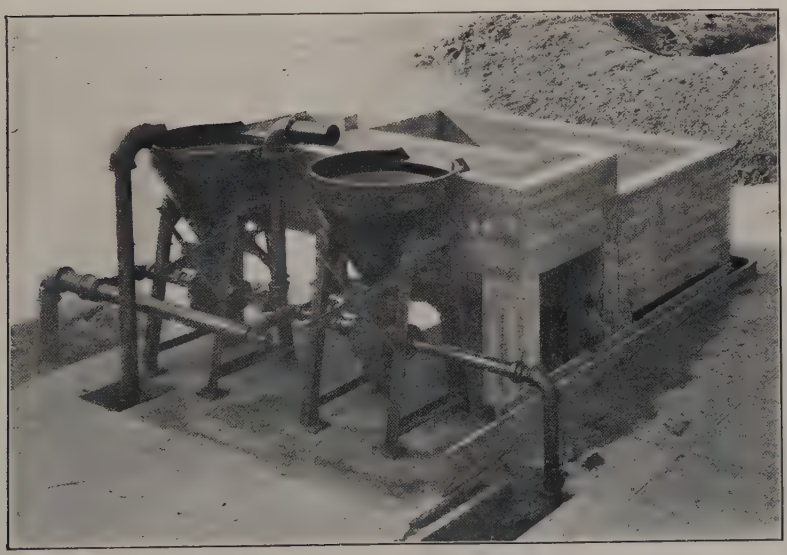


FIG 2.—STATIONARY WASHERS.

upon it from above at considerable velocity. The mixture thus produced contains more sand in proportion to the water, and the ejectors therefore throw more sand and less water. Second, the discharge ends of the ejectors beyond the throats are made with a very flat batter, like the discharge end of a Venturi meter. The batters for the discharge are 1 to 22, in place of 1 in 6, used at Lawrence, Albany, Philadelphia and in other earlier designs. A series of tests showed that with this modification, and with fixed water and sand quantities, twice as much pressure at the discharge could be obtained as with the usual batter on the discharge. In other words, with given quantities of sand and water, it is possible to force the sand through twice as long a line of discharge piping by the use of the flatter batter in the discharge pipe beyond the throat of the ejector. The saving clearly comes from the utilization of the velocity head in the throat, which was largely lost with the greater batter on the discharge.

Water to operate the movable ejectors and all other parts of the sand-washing and handling system is provided under from 80 to 100 lb. pressure by pumps and pipes installed for that purpose. Unfiltered water from the main pump-well is used. The pipes take the water to each court, and 5-in. wrought-iron pipe lines take it down each side of the filters. These have hose connections at intervals averaging about 60 ft. Hose, 2½ in. in diameter, and seldom more than 100 ft. long, completes the feed line. The discharge of mixed sand and water is first through a line of 3-in. hose, seldom more than 100 ft. long, to the nearest side of the filter. Here connection is made with a 4-in. wrought-iron discharge line leading to one of the stationary washers in the courts. The feed and discharge pipes always go together, making a pair, with corresponding connections, and with connections under the vaulting but above the top of the filter walls, so that each pair is capable of serving two filters. The pipes are also laid to grades, with blow-offs, so that when they are not in use they can be entirely drained to prevent freezing.

The design of the pipes for carrying the mixed sand and water was given considerable attention. If they were too small, the capacity would be limited; the workmen would have to wait for the ejectors to carry off the sand that had been shoveled into them; and the apparatus would not prove economical. On the other hand, if the

pipes were too large for the minimum use, the sand would settle out of the water, form a deposit in the pipes, and stop the operation.

By authority of Colonel Miller, tests were made to determine the power and capacity of ejectors of different sizes and types, the frictional resistances of mixtures of sand and water in pipes, and the minimum velocities at which such pipes could be regularly operated without stoppage by separated sand.

In making these tests an ejector was connected by hose to a length of wrought-iron pipe, which finally discharged into a tank on a wooden trestle. The outlet from this tank was through a circular hole in a disc of sheet-iron upon the bottom, which acted as a standard orifice. Pressure gauges connected with the pipe at intervals gave better results as to loss of head than could be secured by differential gauges of glass tubes. There is a throbbing in pressure, particularly at moderately low velocities, perhaps due to a partial deposition and subsequent picking up of sand particles, and to separations into more or less dense mixtures, which made it difficult to get satisfactory readings in the glass tubes. The pressure gauges were less sensitive and therefore much more easily read, and, on the whole, sufficiently accurate.

The time required to throw a measured volume of sand was observed by one man, while others read the gauges on the discharge pipe at 10-sec. intervals. At the same time the head over the standard orifices was noted, and from this the total volume of flow was computed.

Occasionally, the discharge from the orifice was taken by a spout to a tank for a determined interval. The results thus secured were used to compute the coefficient of discharge of the orifice (which was 0.63 for sand mixtures, in place of 0.60 for water, probably because the sand formed a flat cone about the orifice). The sand contained in this water was also often separated and measured, as a check on the rate at which it was passing.

Such experiments were made with 3-in. and 4-in. pipe, from which diagrams of friction were prepared for use in designing the piping for the filters. They are not as accurate as could be wished, and further data of this kind are to be desired.

With an average velocity as low as 2 ft. per sec., a pipe would promptly fill up with sand and shut itself off. This was true, what-

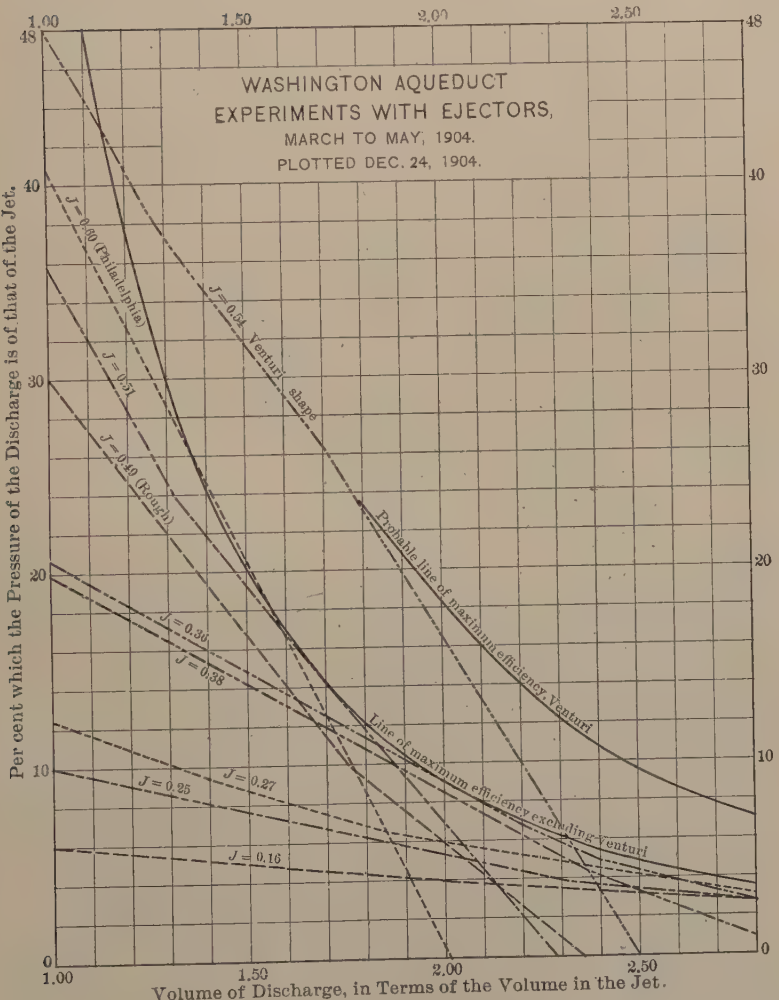

WASHINGTON AQUEDUCT
EXPERIMENTS WITH EJECTORS,
MARCH TO MAY, 1904.
PLOTTED DEC. 24, 1904.
J = 0.30 (Philadelphia)
J = 0.21
J = 0.49 (Rough)
J = 0.34 Venturi shape
Probable line of maximum efficiency Venturi
J = 0.36
J = 0.33
Line of maximum efficiency excluding Venturi
J = 0.25
J = 0.27
J = 0.16
Per cent which the Pressure of the Discharge is of that of the Jet.
Volume of Discharge, in Terms of the Volume in the Jet.
1.00
1.50
2.00
2.50
0
10
20
30
40
48

FIG. 12.

ever proportion of sand was present in the water. At 2½ ft. per sec. flows of sand and water were sometimes maintained, but always with great fluctuations in friction, which, the writers think, indicates a partial deposition of coarse particles and an occasional picking up and carrying forward of an accumulation. Occasionally, at this velocity, the pipe would fill so as to shut off the flow.

At velocities of 3 ft. per sec. the flow was much steadier, and stoppages almost ceased. At 4 ft. per sec. and upward, mixtures of sand and water acted quite as a fluid, and the flow was almost as steady and regular as that of so much water, although the frictional resistance was much greater.

As a result of these studies, and calculations based upon them, it was concluded that:

A short length of 3-in. hose or pipe would not throttle an ejector too closely, but a line of such pipe long enough to serve the most distant positions of the movable ejectors would cut down the rate of discharge to such an extent that the men would have to do too much waiting for the ejectors.

It would never be possible to discharge two ejectors through the same 3-in. pipe.

Adequate carrying capacity is furnished by 4-in. pipe, and one such pipe will usually be capable of taking the discharge from two ejectors, if occasion demands.

To prevent obstruction in 4-in. pipe, the jet of the ejector should be of such size as to throw water enough to produce a velocity of 3 ft. per sec. in 4-in. pipe. In practical operation the sand taken up and the water introduced through the irrigator will increase the volume by one-third, and will produce a velocity of 4 ft. per sec. in the 4-in. discharge, and this is desirable.

With two ejectors discharging through one 4-in. pipe, the velocity in it will be about 8 ft. per sec., which is permissible.

The most advantageous ratio of diameter of jet to throat in ejectors was studied at some length, and sufficient information secured to select a throat diameter of approximately the most advantageous size. A diagram of some of these results was made, which shows the results clearly.

A large throat will throw more sand, but the pressure that can

be produced to overcome friction in the piping decreases almost as the square of the throat diameter. The ratio of jet to throat, as computed for the different ejectors, based on the data of the experiments, varied greatly. For instance, in the movable ejector,

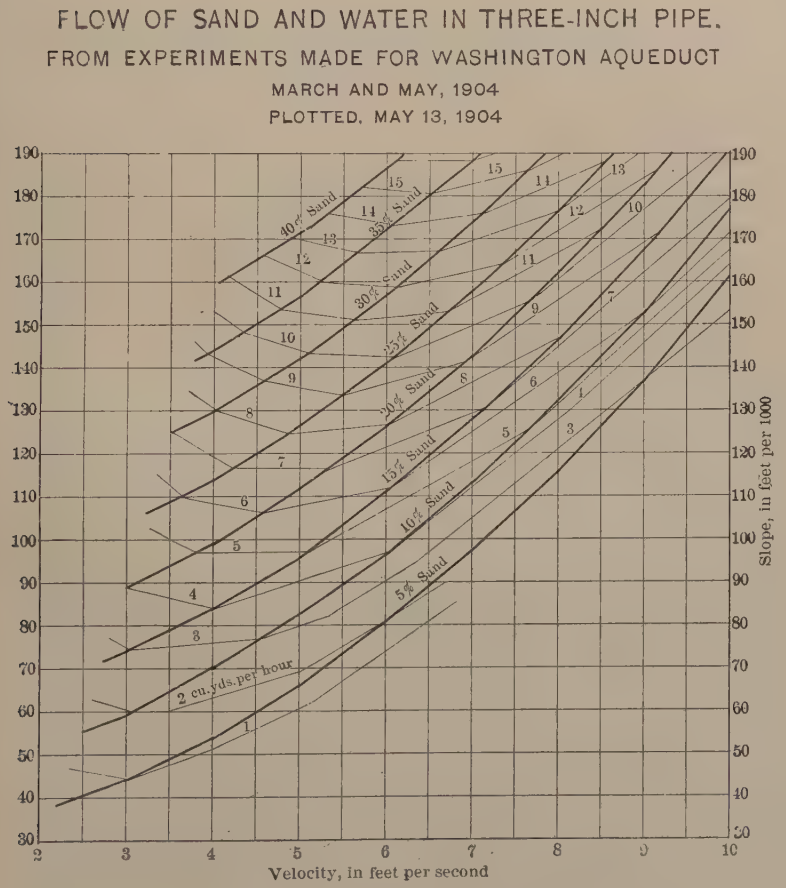


FIG. 13.

the jet is 0.59 of the throat. In the first ejector in the stationary washers, which only throws the sand from one hopper to the next, the ratio is 0.34, and in the second one, which lifts the sand to the sand-bin, the computed economical ratio is 0.48. In all cases the

ratios are changed by wear, and the throats will need to be replaced frequently.

Sand Washers.—The stationary washers in the courts are somewhat like the washers at Albany, but differ in one important particular. The mixed sand and water falls into a hopper, but below this is a chamber formed by a globe casting. There is a free opening from the hopper to this chamber. A secondary jet of water (called an "irrigator") enters this casting near the bottom, and supplies just about the quantity of water which is carried into the throat of the ejector with the sand as it leaves the hopper. By this arrangement there is no downward flow of water from the hopper to the chamber below. The sand settles through the water into the chamber, and is separated from practically all the dirty water which came with it. This overflows from the top of the hopper and is wasted.

In the Albany washers, each hopper serves to dilute the dirty water in the sand which reaches it. In a general way, each hopper serves to reduce the foreign matter by one-half. In the new type there is practically a complete separation, and theoretically a single hopper should serve to wash the stand adequately. As an added safeguard, a second hopper is provided, however, which may or may not be used, as desired.

This type of sand washer was designed by one of the writers for another service,* and has since been used in smaller plants.

The waste water from the hoppers goes through a pair of concrete boxes, to stop such particles of sand as may be carried out of the hoppers, and this sand is shoveled back to the washers from time to time.

The stationary washers and their appurtenances are erected on blocks of reinforced concrete, strong enough so that any movement, either from settlement or frost, will move the block and the apparatus as a whole, and will not move the different parts of it in relation to each other.

The available space in the stationary washers did not permit the use of a batter of 1 in 22 on the ejectors, but a batter of 1 in 14 was obtained. This produces an ejector powerful enough to lift the sand about 26 ft., to the top of the storage bins.

* Described in *The Engineering Record*, January 25th, 1902, and in *Engineering News*, February 13th, 1902.

Sand Bins.—The storage bins are of reinforced concrete, cylindrical in form, with conical bottoms. They are supported on circular concrete foundations, carried well below the frost line, and also below the pipes in the courts. The weight of the bins is carried partly upon an outside wall and partly upon three circular arches which intersect in the center of the structure, leaving a space underneath large enough for a two-horse cart to drive through to be filled with sand from the gate.

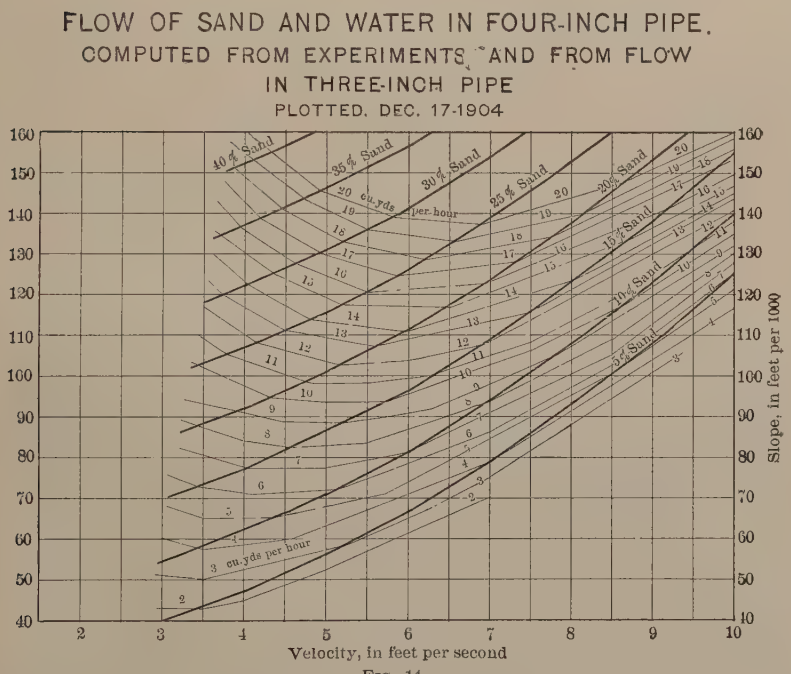


FIG. 14.

Each bin holds 250 cu. yd. as it can ordinarily be filled, which is not quite to the top. The material thrown to these storage bins consists of 1 part of sand and 3 or 4 parts of water. The sand settles to the bottom, and the water accumulates in the bin until the latter is half full. It then overflows through a pipe at this level, and this continues until the sand nearly reaches the outlet. The gate on the first outlet is then closed, and the process is repeated by the use of a second outlet almost at the top of the bin.

Twenty-nine bins are provided, one for each filter, but the connections are such that each filter is tributary to a number of bins. The filters and bins are arranged in three groups. The bins in each group will contain sand equivalent to 19 in. on one filter, or somewhat more than the quantity to be replaced at one refilling of a filter. The plan is to refill a filter as often as the bins are full, and to use all the accumulated sand in one group in doing it. This will be every two or three months.

There may be seasons of the year when it will not be wise to replace the sand in the filters. Experience at Albany indicates that, after replacing sand, the quality of the effluent temporarily falls off somewhat. In summer the change is slight and unim-

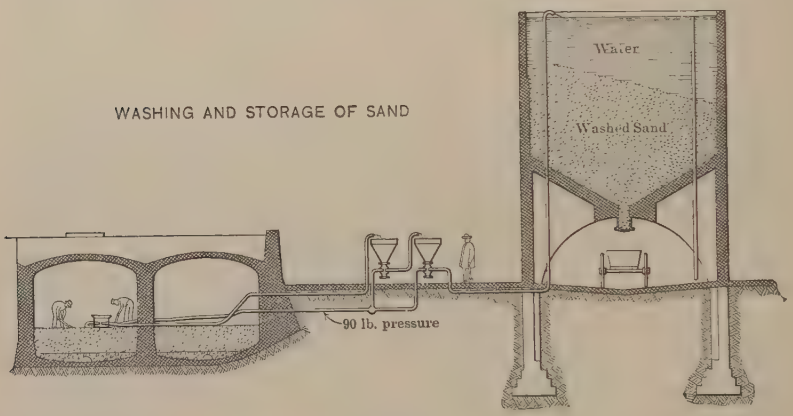


FIG. 15.

portant. In colder weather it is much greater. If this proves to be the case at Washington, refilling will not be advisable in winter; and after the bins are once filled with sand in cold weather, the remainder of the sand removed from the filters will be piled up in the courts until spring. Computation showed that the saving in labor and handling sand would make it pay to build bins where they could be used several times a year, but that it would not pay to build them when they could be used only once a year. No bins are provided, therefore, for this exceptional winter condition, and, as far as it occurs, the sand in excess of the capacity of the bins will be handled by the older methods.

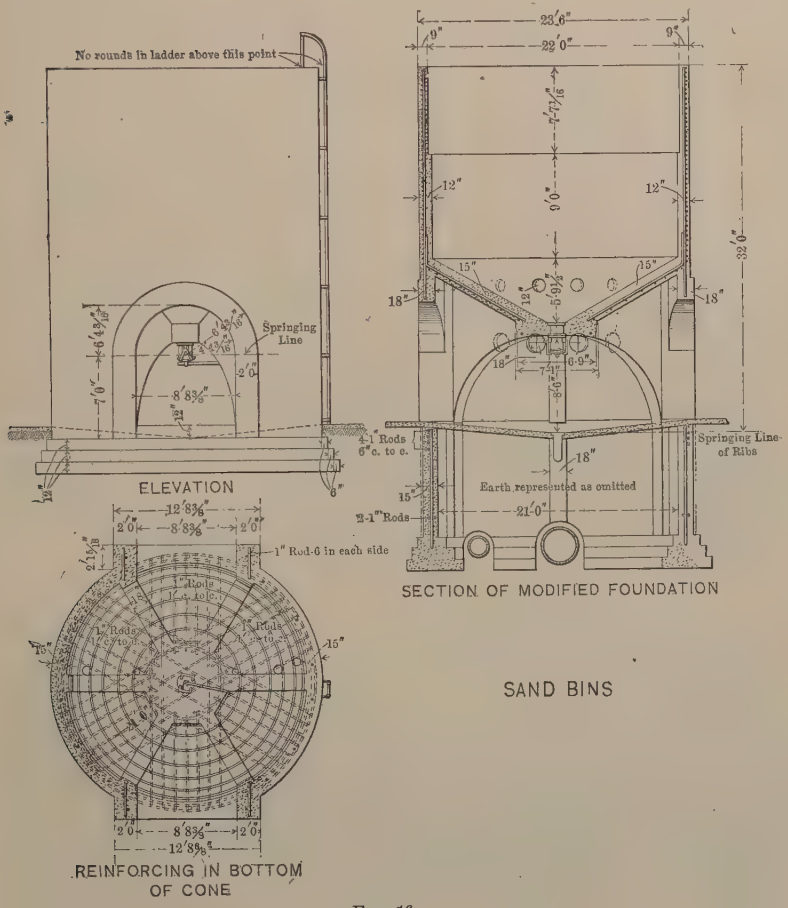


FIG. 16.

The sand handling machinery was completed so that it was in use for the greater part of the work from the last of April, 1906. The figures thus far obtained indicate average costs for labor as follows:

Scraping, 5 cents; leveling, 3 cents	8	cents per cu. yd.
Ejecting	14	"
Washing	4	"
Storing and replacing sand	16	"
Total cost	42	cents per cu. yd.

These figures may be revised, with more extended experience, but the present indication is that the entire cost of sand handling will not exceed 42 cents per cu. yd. The apparatus has fully justified the additional investment, for the cost is far lower than has been yet reached elsewhere.

For comparison, the average costs of sand handling, as computed by George W. Fuller, M. Am. Soc. C. E.,* for some of the older filters, are as follows:

Lawrence, in service, 1893.....	$1.70	per cu. yd.
Mount Vernon, in service, 1894..	1.51	"
Albany, in service, 1899.....	1.38	"

From data in published reports it appears that the cost at Philadelphia, with hydraulic ejecting, is about $1.20 per cu. yd.

It should be stated that the average depth of sand removed at Washington is greater than at the other places, but that the scrapings are less frequent; so that the quantity of sand handled per million gallons is about an average figure.

Pumping Station.—The pumping station serves to lift the water from the Washington City Reservoir to the filters. In the same building are located, also, the pumps which supply raw water under high pressure for washing and transporting the sand; an electric light plant; boilers and other auxiliary machinery. When the Washington City Reservoir is at its normal high-water mark, it is 20 ft. below the level of the water on the filters. To this must be added about 1 ft. for frictional loss in the piping, including the head loss in the Venturi meters. This makes the normal minimum pumping lift 21 ft. The Washington City Reservoir will not always be entirely full, however. When it is drawn down the pumping lift will be increased. It is possible to draw it approximately 14 ft., in which case the pumping lift will be increased to 35 ft. The capacity of the reservoir between these levels is 180 000 000 gal.

The main pumping is done by three centrifugal pumps, directly connected to Harrisburg engines, all furnished by the Henry R. Worthington Company. These pumps at the minimum lift are guaranteed to deliver 40 000 000 gal. per 24 hours. They have ac-

Transactions, Am. Soc. C. E., Vol. XLVI. p. 336.

FIG. 1.—GENERAL VIEW OF A PART OF THE WASHINGTON FILTRATION PLANT.

FIG. 2.—MOVABLE EJECTOR AT WORK MOVING SAND FROM FILTER.

tually delivered nearly 25% more than the guaranty. At greater lifts the guaranteed capacity decreases, so that the product of the capacity and lift is always 840 000 000. The guaranteed duty was 100 000 000 ft-lb. per 1 000 lb. of dry steam. On test the duty developed was 97 500 000 ft-lb.

It was found that there was considerable loss of head in the surface condensers on the discharge, and the contractors are now changing these condensers. The saving made in this way will represent more than the deficiency in the duty, so that the pumps will fully comply with the guaranty.

There are two pumps for the sand wash-water, each with a capacity of 2 500 000 gal. per 24 hours, with 100 lb. pressure. It is expected that one pump operated through one 8-hour shift per day will generally maintain this service. Occasionally, it may be necessary to operate two pumps, or to extend the period of work beyond 8 hours per day. The pumps are compound, designed to stand rapid fluctuations in rate and pressure, and are not designed to give the greatest economy. The duty shown on test was about 65 000 000 ft-lb. per 1 000 lb. of steam, thus exceeding the guaranteed duty of 60 000 000 ft-lb.

The electric-light equipment consists of one 50- and one 25-kw. generator, directly connected to compound steam engines. This serves to light the pumping station, the office and laboratory, and the courts, and furnish current for other incidental purposes. The filters are also wired and provided with electric light throughout. There is one 16-c.p. light for each 14-sq. ft. section. Generally, to clean the filters, the manhole covers will be removed, admitting daylight. The daylight secured in this way is more satisfactory and cheaper than the electric light, but experience in other places has shown that there are dark days, or times when snow and ice are upon the roof, and when it is a great convenience to be able to clean without removing the covers. The electric lights in the filters, therefore, are regarded as for occasional and auxiliary use, and not for regular service.

Steam is furnished by four 200-h.p. Babcock and Wilcox boilers, with Roney automatic stokers and Green economizers. A Custodis stack, 150 ft. high and 5½ ft. in diameter, serves the boilers.

A coal pocket adjoining the boiler-room is of concrete, and of

practically the same construction as the filters. It is 68 ft. square, with an inside height of 15 ft., and will hold 1 700 tons of coal, or four months' supply.

It is not possible to get a track from any railroad to the plant. Coal, therefore, must be brought in carts; and manholes are provided through which the coal is dumped from the driveway into the pocket.

Laboratory.—The laboratory and office building is 36 by 56 ft., with a high, light basement. It is placed near the pumping station, and high enough so that a good general view of the plant is obtained from its windows. The design follows in a general way that of the original laboratory of the Lawrence Experiment Station of the Massachusetts State Board of Health, having a general laboratory with a roof ventilator in the middle, with special laboratories for chemical and biological work on either side. The superintendent's office immediately adjoins, while the filter foremen and gate tenders have an office across the hall, in which many of the routine records are kept. The basement furnishes storage for supplies, and room for experiments on a considerable scale.

The laboratory department has charge of the routine operation of the filters, and is responsible for the results obtained. It must also keep track, and investigate the condition, of the water from Great Falls to the plant, and note the changes that take place in the aqueducts and reservoirs. It is intended, further, to make experiments upon methods of treating the water with reference to improving the processes and the suggested methods of preliminary treatment.

Cost of Work.—The total sum appropriated for the construction of the filters was $3 468 405. Nearly all the work is finished and paid for, although a little finishing up remains to be done.

Taking the payments, as far as made, and estimating the cost of the parts yet to be completed, a statement of the approximate cost of the work has been prepared (Table 2), and this will not differ greatly from the final figures.

FIG. 1.—PUMPING STATION, OFFICE AND LABORATORY BUILDING, AND INTAKE GATE-HOUSE, AT WASHINGTON CITY RESERVOIR.

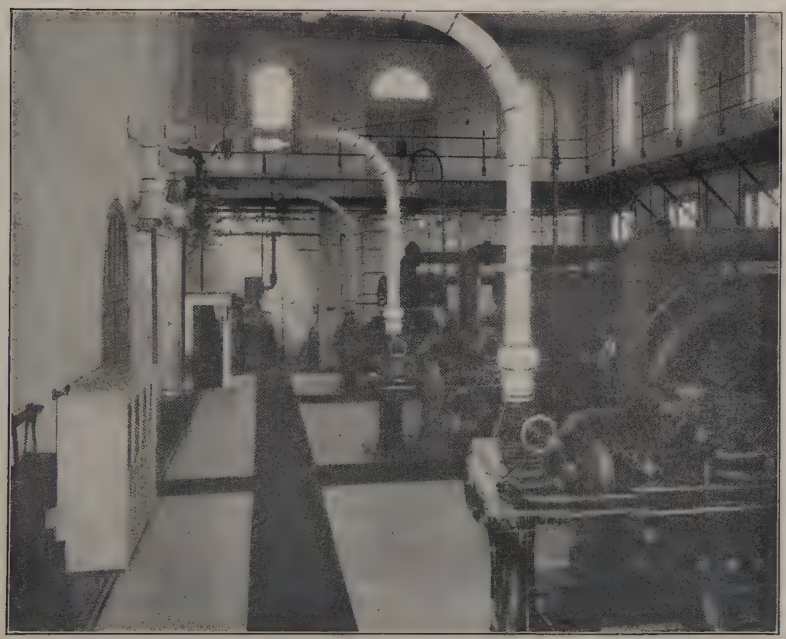


FIG. 2.—INTERIOR OF PUMPING STATION.

TABLE 2.—Approximate Cost of Washington Filtration Plant.

Pumping Station:	
Intake, with gates and building	$11 500
Venturi meters, 72-in. and 54-in. (reinforced concrete)	5 000
Electric lighting, engines, etc.	7 000
Four 200-h.p. boilers, in place	14 800
Four Roney stokers	4 100
Two Green economizers, in place	5 100
Three 36-in. centrifugal pumps, with engines	42 000
Two sand-washer pumps	8 100
Piping, valves, etc.	13 100
Coal, oil, and running tests	3 500
Traveling crane	1 600
Chimney, with foundation	5 800
Coal vault	11 000
Building, with foundation and suction well	51 000
Total cost of pumping station	$183 600
Twenty-nine Filters:	
Excavating, 862 700 cu. yd., at 30 cents	$258 800
Filling, 299 500 cu. yd., at 30 cents	89 900
Sodding slopes, and seeding	7 300
Roads and drains outside of filters	16 200
Concrete tunnel under First Street	3 100
Concrete (including cement).	
Floors, 36 563 cu. yd., at $6.75	246 800
Walls, 19 038 cu. yd., at 7.35	139 900
Piers, 6 964 cu. yd., at 8.25	57 500
Vaulting, 34 920 cu. yd., at 8.75	305 500
Ramps leading to tops of filters	6 800
Court paving (including cement)	43 000
Central underdrains, 7 900 lin. ft., at $1.65	13 000
Interior drainage system, 29 filters, at $500	14 500
Drainage of roofs, 29 filters, at $266	7 700
Materials placed in masonry, 29 filters, at $200	5 800

Doors and beams at entrances	$3 700
Filter sand, 157 725 cu. yd., at $2.65	418 000
Filter gravel, 36 500 cu. yd., at $2.75	100 400
Cast-iron piping and special castings	117 000
Steel rising main, including concrete backing	76 800
Pressure-pipe system	2 600
Sand-washer pipe system, complete	24 000
Sand-washing apparatus, 19 washers, 8 ejectors, and hose connections, complete	4 800
Elevated sand bins, 29, capacity 250 cu. yd. each	60 800
Exterior drainage system	25 300
Venturi meters and indicating apparatus	11 400
Sluice gates and gate-valves	19 900
Regulator-houses, including substructures	27 300
Office and laboratory, with equipment	19 700
Shelter-house for workmen	4 800
Water and gas lines to buildings	11 200
Electric lighting, courts and filters	41 900
Cleaning up and miscellaneous	11 600
Total cost of 29 filters	$2 197 000

Filtered-Water Reservoir:

Excavation, 83 500 cu. yd., at 30 cents	$25 100
Filling, 18 000 cu. yd., at 30 cents	5 400
Concrete (including cement), 15 290 cu. yd. (average) $7.60	116 000
Gate-houses for filtered-water reservoir (superstructure)	3 500
Total, filtered-water reservoir	$150 000

Lower Gate-house:

Pipe lines to lower gate-house	$6 000
Lower gate-house	18 300
Total	$24 300

Land	$619 900
Clerical and Engineering Work:	
General plans	$36 000
Surveying	32 000
Field office force	21 000
Main office force	20 000
Inspecting work and testing materials	67 000
Watchmen	4 500
Temporary office	1 000
Total, clerical and engineering	$181 500

SUMMARY.

Pumping station, including intake, Venturi meters, electric light generating apparatus, concrete, and stack, complete	$183 600
Twenty-nine filters, with excavation, grading, concrete work, piping, regulator-houses, shelter-house, laboratory and office, electric light wiring, sand-washing apparatus, sand bins, etc., complete	2 197 000
Filtered-water reservoir, including excavation, concrete work, gate-house, regulating apparatus, etc., complete	150 000
Lower gate-house and pipe line	24 300
Land	619 900
Engineering and clerical work	181 500
Total cost of work	$3 356 300
Total amount appropriated	$3 468 400
Balance available for operation and contingencies	112 100
Total cost for construction, excluding land	$2 736 400
Percentage for engineering and supervision	6.65

Cost of building per square foot (outside dimensions of brickwork):

Pumping station	$6.70
Regulator-houses (including substructure)	6.63
Shelter-house	3.24
Intake (including substructure)	21.30
Lower gate-house (including substructure)	14.95
Gate-house (filtered-water reservoir)	3.10
Office and laboratory	6.45

Cost per acre for constructing 29 filters:

$2 197 000 ÷ 29 = ... $75 700

Cost per million gallons capacity of filtered-water reservoir:

$150 000 ÷ 14.2 = ... 10 600

Comparison of Costs of Construction at Albany and at Washington.

The Washington filters cost about 65% more, in proportion to their net area, than the Albany filters. Most of this is caused by higher unit prices. Masonry cost 20% more per cu. yd., and on other items there was a general increase of fully 25%, representing the changed business conditions at the two periods of construction. The filter sand cost 2.7 times as much, owing to the supposed difficulty of securing it at Washington, a difficulty which proved to be unfounded.

The costs may be compared as follows:

The masonry was lighter at Washington and the sand thinner, representing a reduction in the whole cost of the work of about	11%
The cost of construction was increased because of the irregularity of the site, and the extra earthwork and filling required, increasing the whole cost of the work by about	16%
Electric lights, sand bins and appurtenances, not used at Albany, were added, increasing the whole cost of the work by about	6%
The unit prices were higher (those of filtering materials making one-half of the addition) increasing the whole cost of the work by about	50%

The product of these factors,

$$0.89 \times 1.16 \times 1.06 \times 1.50$$

is 1.65, or approximately the relative cost of the Washington work.

Operation.—The first filter was put in operation on August 18th, 1905. The effluent from this filter was mixed with the usual supply of unfiltered water in the Washington City Reservoir. One additional filter was started every second day thereafter, until October 5th, when the raw water was cut off and the entire city was supplied with filtered water.

Unfortunately, the laboratory building and apparatus had not been completed at the time the first filter was put into service, but they had reached such a state of completion early in September that samples of the unfiltered and filtered water were plated each day and counts of bacteria made.

Between August 22d and 31st, the turbidity of the water in the reservoir supplying the filters, as determined by the U. S. Geological Survey standard, ranged between 30 and 45 parts per million, and averaged 40. During the first 12 days in September, the turbidity of this water was higher, ranging between 40 and 85, after which it decreased, and was uniformly low until December 8th, when it began to increase again.

The reduction in turbidity of the filtered water was noticeable the very first day after each filter was put into service, and a fair percentage of reduction was invariably shown thereafter.

During October it became necessary to clean some of the filters. On account of the filters having been put into operation before the completion of all their appurtenances, the work of cleaning was considerably greater than would otherwise have been the case. At this time the courts had not been paved, nor had the sand washers and sand-washer pumps been put in condition for use. In consequence, the old method of removing sand in wheel-barrows had to be used, and not only that, but it had to be wheeled on temporary runways to the top of the filters, which added to the expense of scraping, not only for the additional distance and elevation, but for the construction of the runways.

The pumping plant was in the hands of the contractor for a part of the time, and irregularities in service during the tests added to the cost of operation.

Some of the leading data of operation for the first five months are shown in Tables 3, 4, 5 and 6.

TABLE 3.—Turbidity.

Averages by Months.

Month.	Great Falls.	Reservoirs.			Filtered water.
		Dalecarlia.	Georgetown.	Washington City.	
1905					
October	36	21	18	11	1
November	19	19	14	11	1
December	199	84	74	39	6
1906.					
January	94	60	60	52	12
February	45	41	29	22	3
March	272	181	56	46	6
April	167	72	58	46	7
May	56	20	16	10	2
June	303	125	94	41	2
July	130	54	47	43	3
August	375	112	66	45	2
September	85	34	28	25	2
October	127	37	24	21	1
November	27	20	16	13	1

This operation was without coagulant, and the turbidity of the effluent clearly shows the need of it.

TABLE 4.—Bacteria.

Averages by Months.

Month.	Reservoirs.			Filtered water.
	Dalecarlia.	Georgetown.	Washington City.	
1905.				
October			198	78
November			153	27
December	15 637	17 884	3 720	60
1906.				
January		15 719	1 520	39
February	4 086	1 823	562	16
March	1 105	883	654	19
April	1 658	715	399	22
May	206	94	66	17
June	4 628	334	224	17
July	607	486	156	26
August	1 077	1 231	188	14
September	245	136	133	14
October	949	640	268	16
November	1 124	1 176	224	12

TABLE 5.—Pumping Plant.

Month.	Average number of million gallons pumped to filters.	Average lift, in feet.	Average quantity of sand wash-water pumped: 100 lb. pressure.	Pounds of coal burned.	Duty of plant per 100 lb. coal burned.
1905.					
October	67.9	23.3		19 382	68.9
November	66.1	21.6		17 198	69.4
December	67.2	21.5		19 899	60.4
1906.					
January	65.27	21.6		20 659	57.2
February	68.69	21.2		20 264	61.1
March	64.28	20.1		18 494	61.2
April	62.7	21.5		19 212	60.9
May	65.63	21.1	0.6	20 334	62.7
June	67.12	22.8	0.9	23 288	63.9
July	69.18	23.78	1.47	27 151	62.5
August	68.03	24.49	0.64	23 574	66.2
September	69.82	23.89	0.66	25 054	62.0
October	69.14	23.77	1.07	27 043	60.2
November	65.51	21.13	0.30	22 578	55.8

TABLE 6.—Cleaning Filters.

Month.	Number of filters cleaned.	Average duration of runs, in days.	Millions of gallons filtered during run.	Total Cost per Million Gallons.			
				Average cost of cleaning.	Average cost of pumping.	Average cost of office and laboratory.	Average total cost.
1905.							
October	13	49	146	$0.63	$0.90	$0.43	$1.96
November	12	45	136	0.58	0.94	0.53	2.05
December	5.25	51	172	0.40	0.98	0.54	1.92
1906.							
January	4	69	233	0.36	1.38	0.74	2.48
February	2⅔	104	332	0.33	1.14	0.59	2.06
March	4½	130	552	0.51	1.16	0.61	2.28
April	7	149	366	0.76	1.34	0.74	2.84
May	8½	130	298	0.79	1.01	0.59	2.39
June	11	125	297	1.05	1.54	0.59	3.18
July	18.6	70	180	1.57	1.15	0.54	3.26
August	2	49	138	2 57	1.37	0.67	4.61
September	5.5	73	205	1.03	1.36	0.56	2.95
October	23	82	229	1.54	1.14	0.74	3.42
November	7.5	37	120	1.07	1.39	0.62	3.08

The practical operation of the filters has developed one interesting point. The water which reaches them has been settled to an unusual extent. No filters of their type have been used in America with water settled to anything like the extent of the water which reaches them. This has the result of removing all the sand and silt from the water. The residual turbidity which reaches the fil-

ters is entirely of clay, and most of it is in an extremely fine state of division. The water often or usually does not contain particles of a size to form at the surface of the sand the sediment layer usually obtained in the operation of sand filters. The clay particles penetrate the sand to a considerable depth, and gradually fill up the pores in the upper part of the sand layer to a depth of 2 or 3 in. Others go deeper and are deposited in the lower part of the sand layer, while still others go entirely through the filter and appear in the effluent. This condition results in an abnormally long period between scrapings, and in soiling the sand to a most unusual depth. These conditions were fully indicated in the experiments made by Mr. Hardy and Mr. R. S. Weston, under the direction of the late Colonel Miller, and experience in the practical operation of the plant has fully demonstrated the correctness of these experimental results and of the deductions made from them at the time.

The use of coagulant would change this and make the conditions more nearly normal, because the suspended matter in the applied water would be reduced in amount and changed in character, being in the form of a loose floc, capable of being stopped at the surface layer, and thus forming a normal sediment layer.

The plant will cost, complete, less than $3 400 000, and 5% interest and depreciation upon this sum is $170 000 per annum; or, for an average use of 67 000 000 gal., about $7 per million gallons. Adding this to the operating expenses, the total cost of filtering is about $9 per million gallons. As the Government pays an unusually low rate of interest on the money that it borrows, the actual cost is less than this amount.

As to the Proposed Preliminary Treatment of the Water.

Potomac water is substantially free from dissolved coloring matter. It has sufficient alkalinity to allow the reasonable use of coagulant. In hardness it is a fairly soft water, ranging from 58 to 125, and averaging about 93 parts per million, according to the computation of Mr. R. S. Weston.

The sewage pollution, which is believed to have been a material element in producing typhoid fever in the city, is the most important characteristic of the water, from the standpoint of public

health, while the suspended matter or turbidity is that quality of the water which makes the greatest impression upon the public.

The filters as constructed should easily remove the objectionable results of the sewage pollution, so as to make the water entirely satisfactory from a hygienic standpoint. It is clear that they will not always be able to produce a water free from turbidity.

As now arranged, it will be necessary to operate them without coagulating, and to produce as good an effluent as possible in that way.

Consumption.—An important element in the results that can be obtained in this way is the consumption of water. At the present time Washington is using more than 200 gal. per capita per day. A considerable quantity of water is used in public buildings, corresponding to manufacturing uses in other cities, but, after making all possible allowance for every use, the only conclusion that can be drawn is that a large part, probably fully one-half of all the water supplied, is wasted through leaky pipes and fixtures, and without benefit to anyone. The measures to be taken to restrict this waste and reduce the consumption to a quantity approximating the actual requirements are well known to water-works men. It requires only the necessary courage and determination, on the part of those who make the laws and are responsible for their enforcement, to carry out the reform and reduce the consumption to a reasonable amount. If this were done, and if the use of water were reduced to 50 000 000 gal. or less per day, there would be a good prospect of always delivering water substantially free from turbidity.

This could be done, because, in the first place, the rate of filtration would be low. In the second place, the frictional resistance of the aqueduct would be small, and the water in the reservoirs could ordinarily be at the highest permissible levels, and the sedimentation obtained would be considerably more complete than with a larger consumption; but, most important of all, it would be possible to close the intake at Great Falls in times of exceptional turbidity of the river water, and maintain the supply from the reservoirs, which would be large enough to serve for such lengths of time as extreme turbidity lasts. In this way, the average amount of suspended matter in the water taken for treatment would be very greatly reduced.

With the consumption at or near 75 000 000 gal. per day, the

full capacity of the aqueduct, it is impossible to carry the reservoirs entirely full, owing to the frictional resistance in the aqueduct; and, with the reduced quantity of water in the reservoirs and the increased consumption, it is entirely impossible to maintain the supply during the longer periods of turbid water which occur in winter and spring. Under these conditions, therefore, even with the best management, there will be times when effluents with seriously objectionable turbidity will be produced.

To increase the quantity of the supply beyond 75 000 000 gal. on the present basis may be regarded as an impossibility, as this figure represents both the capacity of the aqueduct and substantially the capacity of the filter plant, although both could be forced somewhat beyond it for a few days in case of emergency.

Coagulation Works.—As coagulation works have always been regarded professionally as essential, a brief consideration of the best location and arrangement of such works may be proper.

Location.—Works should be located on the line of the conduit below the Dalecarlia and above the Georgetown Reservoir. The application of coagulant at this point would be more advantageous than at Great Falls, and the Distributing Reservoir is better adapted to receiving the precipitate produced by it than either of the other reservoirs.

Arrangement.—The best arrangement of apparatus for applying the coagulant would be substantially that recently installed at St. Louis, namely, to allow a constant unmeasured stream of water to flow through dissolving tanks on its way to the point of application. Weighed quantities of solid coagulant would be placed in the boxes at short intervals, according to requirements.

Cost.—The cost of constructing the necessary works for adding coagulant would be very small, probably not more than $5 000. The cost of operation would be greater, amounting, as was first estimated, to an average of 50 cents per million gallons for all the water treated throughout the year, although varying considerably in different years with the amount of turbidity. The cost of operation would not be increased by this amount, for the coagulation would clarify the water to such an extent that longer runs could be obtained on the filters, and the cost of cleaning them would be materially reduced.

Improvements in the Georgetown Reservoir.—In connection with coagulation, the Georgetown Reservoir should be reconstructed and furnished with a concrete bottom, laid so as to drain to waste outlets to allow the considerable quantities of sludge, deposited as the result of the coagulation, to be easily and cheaply removed. It should also be baffled, in order to increase its efficiency as a sedimentation basin. These improvements would cost about $100 000. Neither the Dalecarlia nor the Washington City Reservoir is suitable for use as a coagulating basin, as their bottoms and location do not permit of an arrangement for the economical disposal of sludge.

Increasing the Capacity of the Plant.—With these improvements carried out, a comparatively small further improvement would put the plant in such shape that it could be forced to a considerably greater capacity. A pumping station at Dalecarlia Reservoir, arranged to lift the water 5 or 6 ft. could be built at one side of the present channel, and without interfering with the regular use of the works. This would increase the hydraulic slope on both ends of the aqueduct system, so that 100 000 000 gal. per day would pass through it. With the coagulation of all very turbid water, it would be perfectly safe and proper to operate the filters at a rate of 4 000 000 gal. per acre, at which rate the above-mentioned quantity could be treated. The pipes and connections at the filters throughout are sufficient for the treatment of this quantity. There is, therefore, a possibility of increasing the capacity of the plant by one-third at a comparatively small expense.

DISCUSSION.

Mr. Longley.

FRANCIS F. LONGLEY, JUN. AM. SOC. C. E. (by letter).—The first year of operation of the filtration plant described in this admirable paper by Messrs. Hazen and Hardy, has brought to light some unexpected facts regarding the typhoid fever conditions in the District of Columbia; facts which at first sight seemed most unfortunate, but which in the end may lead to a clearer knowledge regarding the epidemiology of typhoid fever in this and other cities.

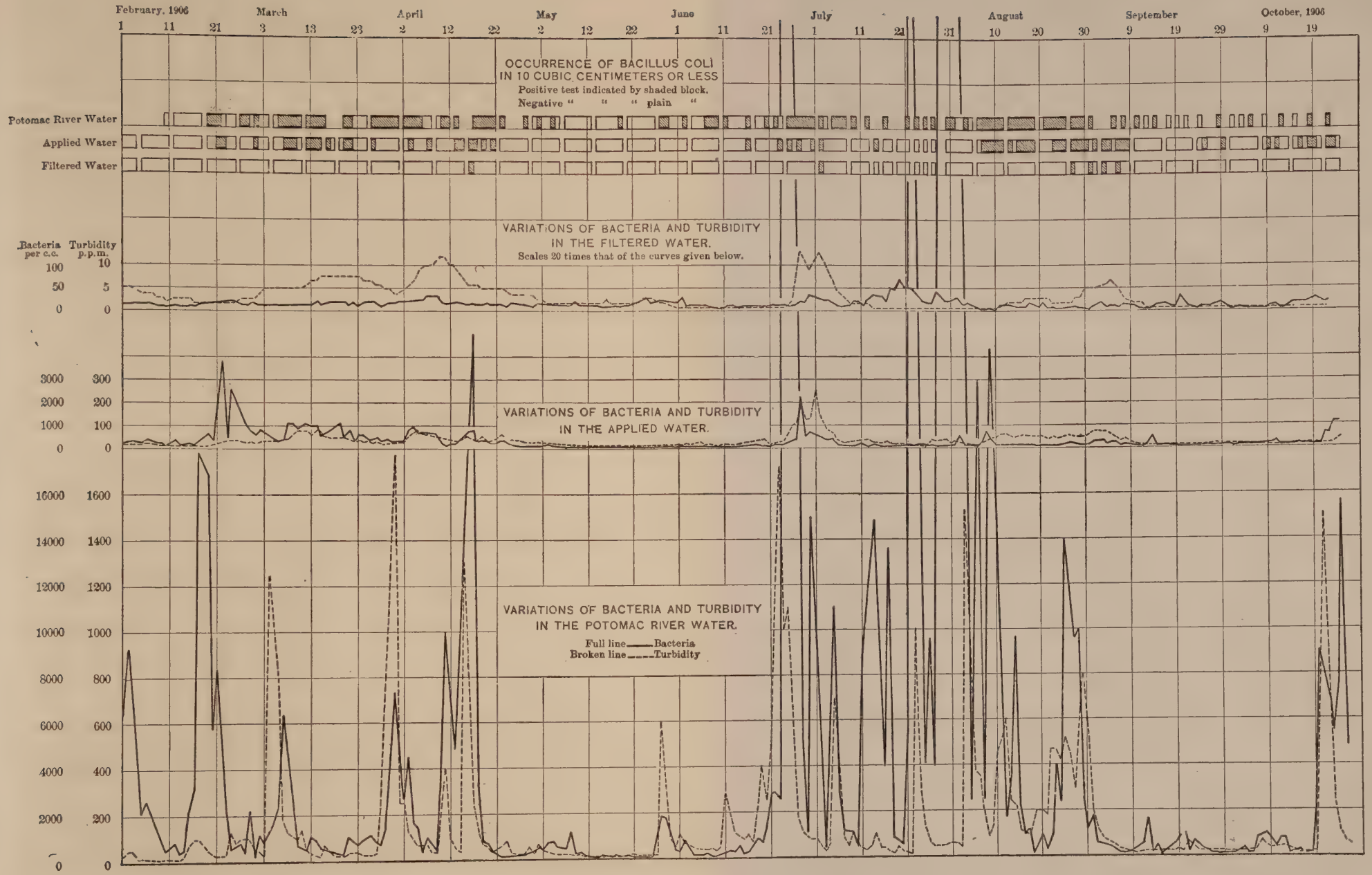
The writer came to Washington to take charge of the laboratories of the filtration plant when it was first put in operation, in the summer of 1905. From the first, certain facts seemed to be evident, in the study of the unfiltered Potomac water. The numbers of bacteria varied with the quantity of turbidity in the water. After a rain, the numbers were very high. As the turbidity following the rain subsided, the numbers of bacteria decreased also, until after a prolonged period of dry weather, when the turbidity had reached its minimum, the number of bacteria in the water was remarkably low. For a period of 40 days in May and June, 1906, and another period of 13 days in September, 1906, all the daily samples of water applied to the filters showed bacterial counts of less than 100 per c.c.

An examination of the results of tests for *B. Coli* also shows some significant facts. Of all the daily samples examined from the applied water, only 11% show the presence of *B. Coli* in 1 c.c., and only 27% in 10 c.c. When the bacterial count is at its lowest, the *Colon Bacillus* may not be detected for weeks at a time, and it is most often present when the water has a high bacterial count during the period of high turbidity following a rain. This variation of bacteria and turbidity, and also the presence of *B. Coli,* are shown on Plate XXXIX.

The inference to draw from these facts is that by far the greater part of the bacterial content of the Potomac water, as it flows into the Washington Aqueduct system, comes from surface washings, clay, soil, manure from the fields, and all kinds of organic and inorganic matter. It is quite evident that constant sewage pollution is not present in appreciable quantity.

The authors give a tabulated statement of the population upon the water-shed above Washington, from which it may be seen that the danger to the community from drinking even the unfiltered water is much less than at Lawrence, or Albany, or Philadelphia, which cities have urban populations on their water-sheds many times as great. There is no intention of arguing that the unfiltered Potomac water is a safe water for domestic use. All unprotected surface waters are subject to specific infection from time to time, and this one is certainly not exempt, as is shown by the close relation traced

PLATE XXXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVII, No. 1086.
LONGLEY ON
WATER FILTRATION
AT WASHINGTON, D. C.



in 1890 between typhoid fever in Cumberland, Md., and in Washington, and reported by Dr. George M. Kober, of Washington, before the International Medical Congress at Berlin. Mr. Longley.

The authors have referred briefly to the satisfactory sanitary quality of the filtered water. With regard to the numbers of bacteria in the filtered water, as determined by standard methods, there is small room for improvement. More significant than the direct reduction of numbers of bacteria in the process of filtration is the exclusion, more or less complete, of sewage organisms. This is truly the fundamental function of a filter; and in attaining this end it is often much simpler—and almost universal practice—to use, as a criterion of the sanitary efficiency of a filter, the reduction in the numbers of bacteria on the assumption that that reduction will likewise be a measure of the decreased risk from pathogenic organisms having their origin in sewage. The routine work of the laboratory of the Washington filtration plant has included careful tests for *Bacillus Coli communis,* the most typical of the sewage organisms, and in Table 7 is given a summary of the results of these tests.

TABLE 7.—SUMMARY OF RESULTS OF TESTS FOR *Bacillus Coli.*

		Total samples examined.	Number positive.	Percentage positive.
Great Falls or Dalecarlia Inlet	10 c.c.	164	71	43
	1 c.c.	164	45	27
	$\frac{1}{10}$ c.c.	164	27	17
Dalecarlia Outlet	10 c.c.	211	97	46
	1 c.c.	211	55	26
	$\frac{1}{10}$ c.c.	211	20	9
Georgetown Reservoir	10 c.c.	177	58	33
	1 c.c.	177	34	19
	$\frac{1}{10}$ c.c.	177	9	5
Washington City Reservoir. (Applied water)	10 c.c.	242	66	27
	1 c.c.	242	26	11
	$\frac{1}{10}$ c.c.	242	3	1
	100 c.c.	148	26	18
Filtered-Water Reservoir	10 c.c.	244	11	4
	1 c.c.	244	5	2
	$\frac{1}{10}$ c.c.	93	0	0
Tap water from various parts of city	10 c.c.	171	8	5
	1 c.c.	171	3	2
	$\frac{1}{10}$ c.c.	59	0	0

In considering the effect of filtered water upon the public health, the significance of the occasional occurrence of *B. Coli* becomes of some importance. Table 8 indicates the presence of *B. Coli* in percentages of total numbers of samples of filtered water examined, for several water supplies.

The effect of filtration upon the water supply, as measured by the improvement in the public health, of all these cities but Washington, and of many others, is too generally known to need any comment.

Mr. Longley. The tests to determine the effect of filtration upon a water are of such a nature as to exclude the probability of serious error in arriving at the percentage of reduction of bacteria and the elimination of *B. Coli* from the effluents of filters.

TABLE 8.—PERCENTAGE OF SAMPLES SHOWING THE PRESENCE OF *Bacillus Coli*.

	Bacillus Coli in:			
	100 c. c.	50 c. c.	10 c. c.	1 c. c.
Lawrence, Mass., 1905	57.9	.....	.....	5.0
Philadelphia, Pa., 1905:				
Upper Roxborough Plant	.....	18.0	.....	.....
Lower Roxborough Plant	.....	26.0	.....	.....
Belmont Plant	.....	34.0	.....	.....
London, England, 6 months ending April 30th, 1906, Thames filtered waters	17.9	.....	8.5	1.9
Washington, D. C., 10 months ending October 1st, 1906, Potomac filtered water	17.0	.....	3.0	1.6

In view of the excellent results obtained, as shown by the laboratory examinations, it was expected that there would be a reduction of the death rate from typhoid fever in the District of Columbia, following the improvement of the supply. The diagram, Fig. 17, will show that this reduction has not occurred.

That the District of Columbia has not an enviable position, in this regard, is shown by Table 9, compiled from the mortality reports of the United States Census.

TABLE 9.—ANNUAL AVERAGE NUMBER OF TYPHOID DEATHS PER 100 000 POPULATION, FOR THE FIVE YEARS, 1900 to 1904, INCLUSIVE.

City.	Deaths.	City.	Deaths.
Pittsburg, Pa.	137.6	San Francisco, Cal.	28.2
Allegheny, Pa.	103.4	Buffalo, N. Y.	28.0
Columbus, Ohio.	65.8	Scranton, Pa.	23.3
Washington, D. C.	62.8	Boston, Mass.	22.4
Cleveland, Ohio.	58.7	Detroit, Mich.	21.9
Louisville, Ky.	58.5	Paterson, N. J.	21.7
Cincinnati, Ohio.	55.8	Providence, R. I.	21.3
Philadelphia, Pa.	49.6	Newark, N. J.	20.1
Kansas City, Mo.	48.8	Fall River, Mass.	19.8
Indianapolis, Ind.	48.2	New York, N. Y.	19.2
Denver, Colo.	47.0	Omaha, Nebr.	19.1
Memphis, Tenn.	45.5	Syracuse, N. Y.	18.7
New Orleans, La.	44.9	Jersey City, N. J.	18.6
Minneapolis, Minn.	44.7	Milwaukee, Wis.	17.1
New Haven, Conn.	44.5	Worcester, Mass.	16.0
St. Louis, Mo.	39.3	Rochester, N. Y.	14.9
Baltimore, Md.	36.5	St. Paul, Minn.	14.8
Toledo, Ohio.	34.7	St. Joseph, Mo.	13.9
Chicago, Ill.	29.7		

Mr. Longley.

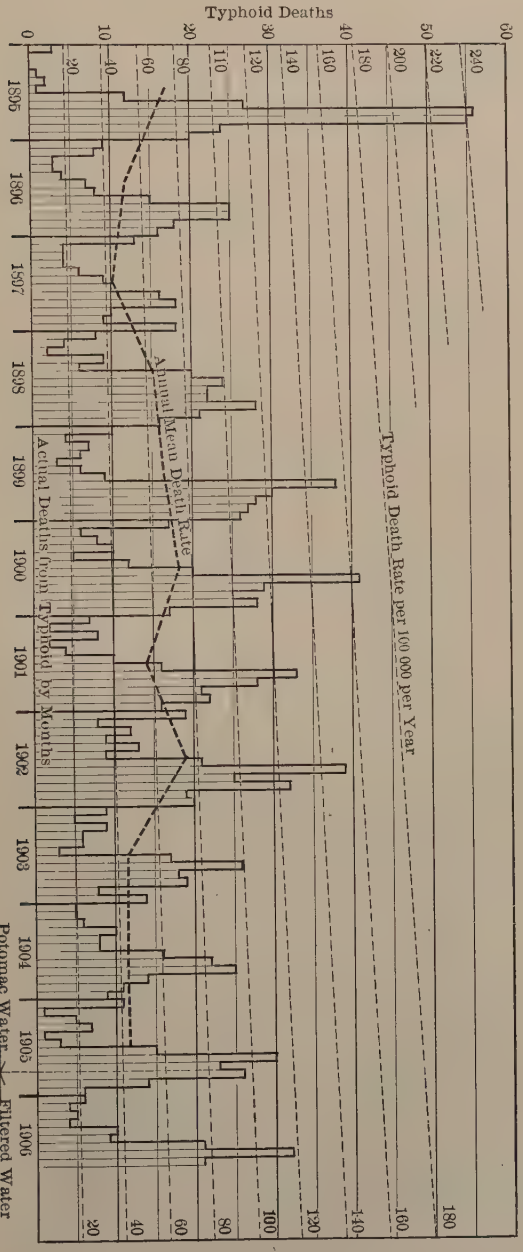


Fig. 17.

Mr. Longley.

The condition of the filtered-water supply has already been set forth. In addition to this, there is another supply which is of interest in this discussion. In the District of Columbia there are more than 100 public wells, most of which are classed as "shallow wells," having depths varying from 7½ to 40 ft. These wells are very freely used, especially during the summer months, when the high temperature of the city supply makes it unpalatable. These wells have all been examined, and, on the strength of the analyses, they fall naturally into two groups, "deep wells" and "shallow wells," quite different in character. The water from the deep wells is quite generally of good sanitary quality. Chemical and bacteriological examinations are not at all necessary to show that the water from the shallow wells is bad. A great many of the wells are very close to the sewers; in fact, on the drawings in the office of the Sewer Department, some of the wells seem to be directly on the line of the sewers. The ground-water level is practically everywhere below the level of the sewers, so that any leakage must be from the sewers to the ground-water.

This condition of the shallow wells is brought out very clearly by the examination of the water. Chlorines and nitrates are all extremely high, and the *Colon Bacillus* was found in 25% of the wells. In view of these results, and more especially in view of the close proximity of wells and sewers, wastes, and drainage of all kinds, it is safe to say that none of these shallow wells can be used continuously without being a menace to the health of the District.

There are, of course, a number of other factors to be considered as affecting the typhoid death rate. The milk supply is to be regarded with some suspicion, and with good reason, as some 10% of the cases during 1906 have been traced quite definitely to milk infection. It is hoped to remedy this part of the situation during the coming winter by legislation providing for better and more extensive inspection service.

The ice supply of Washington is probably no worse than the average. To quote from a classic work by Professors Sedgwick and Winslow on the subject of ice:

"The general result of human experience, the absence of epidemics of typhoid fever traced conclusively to ice, the fact that cities like New York, Lowell, and Lawrence have used ice of polluted streams, and have yet maintained low death rates from typhoid fever, all tend to support the conclusion at which we have arrived, namely, that natural ice can very rarely be a vehicle of typhoid fever."

Barring the possibility of some extraordinarily unsanitary conditions existing in some of the local ice factories, the ice supply may probably be regarded as having no important relation to this question.

Mr. Longley.

The question of infected shell-fish is probably of small importance in this problem, as the maximum typhoid death rates come at the time of year when the consumption of raw shell-fish is at its minimum.

It can readily be conceived that contaminated fruit and vegetables may have much to do with the presence of typhoid fever in Washington. An apple or a head of lettuce may pass through devious routes and through many questionable hands before reaching the consumer, who eats it with wonderful faith in the cleanly habits of his kind. A great deal of garden truck and small fruit sold in the markets of Washington is grown not far from the city. Typhoid fever has prevailed in this region for many years, and may easily take this course in keeping up the endemic condition. The writer has made examinations of quantities of fruit and vegetables, and has found suspicious organisms in surprisingly large numbers at times. It does not seem improbable that a good deal of infection might be spread by this means, going to all classes, in all parts of the city, at just the time of year when these commodities are most plentiful in the market.

The colored population of Washington offers a good field for study of conditions which favor the spread of the disease by contact. The relative typhoid fever mortality among the white and colored population has been the subject of some study. The colored people as a class are more ignorant of the principles of hygiene than are the whites. Their circumstances and environment are less favorable to health. They are more careless in their personal habits. As a race, they are more susceptible to disease than the whites; this statement is substantiated by the fact that the health reports of the District of Columbia show greater death rates among the colored population in twenty-six out of thirty-one causes of death cited. And perhaps not the least important is the fact that they are much more gregarious than the white population, which facilitates the spread of communicable diseases among them.

In Table 10 are given the typhoid death rates of the white and colored populations of several cities. These are based on a total population equivalent to about 4 000 000 colored people for one year, and a much larger number of white people. Other figures are at hand, less reliable, but all substantiating those given herewith.

The figures in Table 10 show in every case a higher typhoid death rate among the colored than among the white population, and it is reasonable to suppose that the influence of this higher rate will be felt to a certain degree among the white population with whom they mingle.

Of course, the white population in itself is subject to infection from this cause, too. As long as people remain untaught as to the necessity for, and the most efficient means of, proper disinfection, the

Mr. Longley.

TABLE 10.—TYPHOID DEATH RATES PER 100 000 POPULATION PER YEAR.

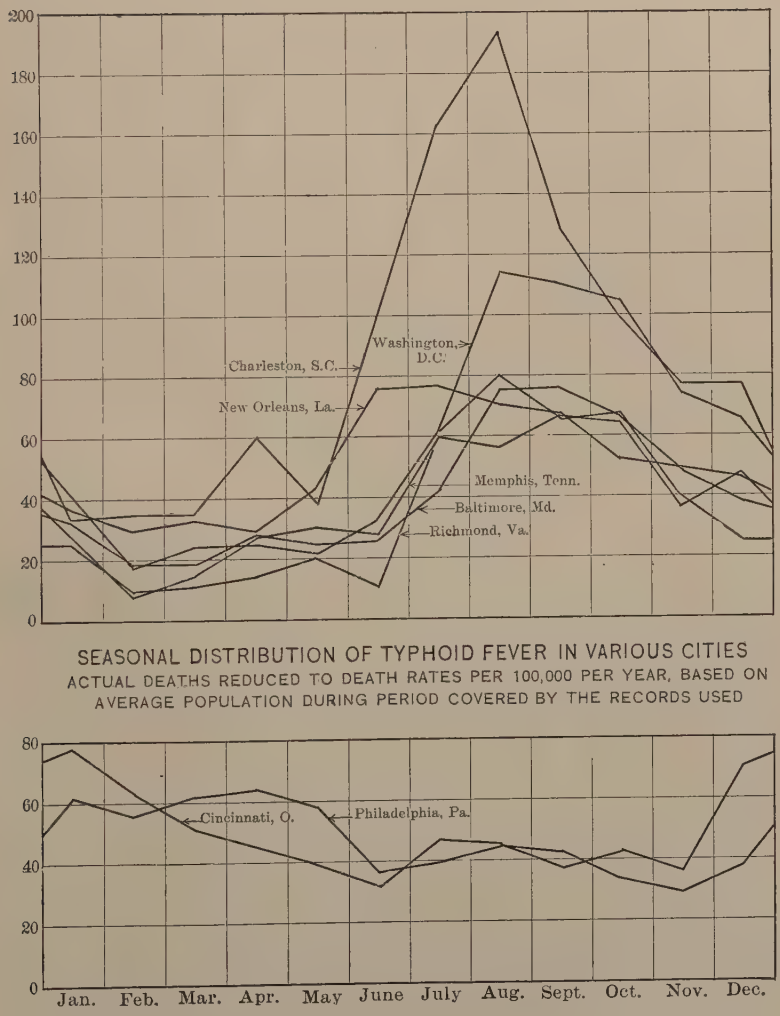
City, etc.	White.	Colored.	Ratio, colored to white.
Cincinnati, Ohio	47.2	94.0	1.99
Washington, D. C.	60.5	88.4	1.46
Baltimore, Md.	42.1	57.1	1.36
Charleston, S. C.	76.0	94.8	1.25
New Orleans, La.	47.5	58.7	1.24
Richmond, Va.	34.6	37.9	1.10
Memphis, Tenn.	37.7	40.0	1.06
AND FROM THE REPORT OF THE 12TH CENSUS:			
The United States (entire area, for the year 1900)	43.0	73.9	1.50
The United States (registration area, for the year 1900)	32.4	69.5	2.15

progress of the disease from this cause will be inevitable. And there is probably little doubt that a great deal of the typhoid fever everywhere is carried in this way, from one person to another, in some cases directly, but more often by ways too devious and complicated for even the most painstaking observer to trace.

In considering the factors that may have an influence upon the presence of typhoid fever in the District of Columbia, an examination of the vital statistics of certain other cities may bring out information of some value. A number of cities have been studied. They were chosen with regard for their similarity to Washington in a good many respects. Leaving out of consideration the large floating population which is of necessity a part of the capital city, Washington is distinctively southern. It has a larger colored population than any other city in the country. Most of these cities have the large open markets typical of the South; and most of them are exposed to the dangers from many polluted wells within their limits. As to their public water supplies, five of them, Philadelphia, Baltimore, Richmond, Cincinnati, and New Orleans, are like Washington in having surface supplies subject to pollution. The other two—Memphis and Charleston—have Artesian supplies which, as far as they go, may probably be assumed to be above reproach.

For purposes of comparison, the typhoid deaths, for each month and for each city, have been reduced to death rates per 100 000 population per year, based on the average population during the period covered by the records used. This seasonal distribution is shown in the diagram, Fig. 18. The figures are equivalent to observations upon a population of 20 000 000 people for a period of one year.

The most obvious characteristic of all but two of these curves is the similarity to the annual temperature curve, as has been pointed out by Professors Sedgwick and Winslow in a paper on the seasonal



SEASONAL DISTRIBUTION OF TYPHOID FEVER IN VARIOUS CITIES

ACTUAL DEATHS REDUCED TO DEATH RATES PER 100,000 PER YEAR, BASED ON AVERAGE POPULATION DURING PERIOD COVERED BY THE RECORDS USED

FIG. 18.

Mr. Longley.

prevalence of typhoid fever. Their general observation was that, in cities having comparatively pure public water supplies, this curve of seasonal distribution of typhoid fever was similar to, and followed a little after, the temperature curve; while in cities having badly polluted public water supplies, the curve was very irregular, bearing no constant relation to the temperature, and having in some cases several maximum points in a year.

On this assumption, these cities may be grouped in two classes: Cincinnati and Philadelphia have curves typical of polluted supplies, and the other cities have the characteristic curves following the temperature curve. That Philadelphia has had a badly polluted supply is conclusively proven by the remarkable reduction in the typhoid death rates in those parts of the city served by the new filtered supply. The evidence is strong, too, that Cincinnati's supply is badly polluted. The irregularity of the curves for these two cities is much more evident from an examination of the curves for successive years than from the mean curve shown in Fig. 18.

On the other hand, two of the cities of the other group have Artesian public supplies which may probably be considered above criticism, from a sanitary point of view. The others have surface supplies subject to pollution, but in none of them are the conditions such as to make the water supply necessarily a serious and constant menace. Therefore, in view of the similarity of conditions in these cities, and of the curves of seasonal distribution of this disease; and in view of the dissimilarity to the two cities of the other group, is it not readily conceivable that the factors tending to produce typhoid fever in the six cities of this class should be much the same; and that the case of Charleston, with its public water supply eliminated from the question, might be the same as of Washington, only in an exaggerated form?

A peculiar feature of these curves suggests an unanswered question. The lowest monthly death rates, in the cities having curves typical of "polluted supplies," come in August, September, and October, when the death rates in the other group of cities are highest. If it be assumed that these two cities simply have these two factors in addition to the factors common to all the cities, why do they not have a high curve during these late summer months, in addition to their "polluted water" curve?

In seeking reasons for the present prevalence of typhoid fever in Washington, in amount as great as in previous years, one suggestion has been made which is full of possibilities, and which time will either prove or disprove. What might the typhoid death rate have been during this year if the filtration plant had not existed? It is quite conceivable that the typhoid due to the use of Potomac River water might have been eliminated more or less completely, and that this beneficial effect might have been entirely obscured by an abnormal quantity of the disease due to some other cause. It

has been common experience in many cities that the typhoid death rate does not go down to bed-rock immediately after the improvement of the supply. The decrease often extends through a term of years, and a comparatively slight abnormal increase in the first year due to other causes might easily cover up the decrease due to the use of better water. This, of course, is mere conjecture, as no evidence has been found tending to corroborate it. Mr. Longley.

To sum up, Washington has had an excellent filtered water supply for more than a year; the typhoid rate has not been reduced in the least; the milk supply is known to be the cause of some of the typhoid; considerable suspicion may be cast upon the many shallow wells, and also upon market products eaten in their raw state; undoubtedly much infection is spread by contact, and probably some by the agency of flies. These and other possible factors are so mingled that the problem is a very complicated one even to those most familiar with all the conditions.

These various factors are touched upon in only a very superficial manner. They are not dismissed as settled, in any degree, as many of them are conditions about which the writer has but imperfect knowledge. The problem is being investigated at length by competent authorities, and their report, when made public, will throw much more light on the subject than does this discussion. It is greatly to be hoped that the results of the investigation will point the way for definite remedies, and at the same time give a better insight into the conditions under which typhoid fever exists in many of the cities of the United States.

WILLIAM C. WOODWARD,* M. D. (by letter).—In the present situation and in this paper the writer has a special interest, as he was one of the representatives of the medical profession of Washington in the efforts that led, in the first place, to the determination to purify the water supply, and in the second, to the use for that purpose of the slow sand process rather than the chemico-mechanical method. Lack of proper technical training prevents him from discussing the paper directly, and he will merely submit a few queries and a few facts which may lend point to the discussion by others. Dr. Woodward.

In the first place, it is stated that the sand used at Washington was required by the specifications to have a greater freedom from fine particles than the sand used with such satisfactory results elsewhere. This, in effect, means that it was desired that the interspaces between the grains should be larger, and that the filter was designed with that end in view. Why, then, when the excessively fine particles suspended in the applied water find their way further into the sand layer than is usually the case, should it be attributed to the peculiar composition of the applied water, instead of to the unusual character of the sand layer?

* Health Officer, District of Columbia.

Dr. Woodward. Again, it is stated that the water reaching the filters has been settled to an unusual extent, and that no filters of the same type as those at Washingon have been used in America with water settled anything like so much. This, it is said, is the cause of that peculiar condition of the applied water which is responsible for the penetration of suspended matter so deeply into the sand, and a project is submitted for the installation of apparatus for adding a coagulant to the water and for transforming one of the reservoirs into a more efficient sedimenting basin, in order to overcome the difficulty. The question naturally arises, why is it necessary to permit such excessive sedimentation to continue? Are not the reservoirs provided with by-passes, and cannot the extent to which the water is sedimented be accurately controlled within reasonable limits by such by-passes? And if such by-passes have been used and the unsedimented water has been applied to the filters without satisfactory results, why attribute to the excessive sedimentation the conditions complained of?

Moreover, has it been demonstrated that, as a matter of fact, the penetration of the fine clay into the sand layer for a distance of 2 or 3 in. does harm? Apparently, it renders it necessary to remove more sand at each scraping, but does it not proportionately diminish the frequency of the scrapings? Have not the freshly scraped filters been found as efficient as any other freshly scraped filters? And has not the bacterial efficiency of the Washington filters been equal to that of any filters where the sediment penetrates the sand less deeply?

It is stated that the use of a coagulant would result in conditions more nearly approaching what are termed normal, because the suspended matter in the applied water would be reduced in quantity and changed in character, being in the form of a loose "floc," capable of being stopped at the surface layer. If the excessive sedimentation of the applied water is now responsible for the penetration of the suspended matter to an unusual depth into the sand, and such penetration is objectionable, is anything to be gained by still further reducing the quantity of suspended matter by using a coagulant? And is it so certain that the quantity of matter suspended in the water when the surface of the sand layer is reached would be reduced? The quality of the raw water is subject to variations which are neither infrequent nor closely limited to their range, and the quantity of water used also fluctuates from time to time. Under such conditions, can the chemistry and mechanics of coagulation and sedimentation be controlled so as unerringly to throw out of suspension a quantity of sand and clay in excess of the quantity of precipitate that will reach the sand layer?

And finally, is it so certain that the cost of operating the system

with the use of coagulants would not be materially greater than it is now? At the very beginning there is an outlay of $105 000 for the necessary apparatus for admitting the coagulant and for remodeling one of the reservoirs to meet the changed conditions. And if, to the interest and sinking fund based on this investment, there is added the cost of labor and material, the total annual cost of the coagulating works would be very considerable. What evidence is there to show that this would be offset by a reduction in the cost of cleaning the filters? Would the collection of the precipitate on the surface of the sand permit such an extension of the length of the runs as the authors of the paper anticipate? The arresting of this flocculent precipitate on the very surface of the sand might result in a denser layer, it would seem, one more nearly impervious to water, and thus necessitate more frequent cleaning, and increase the total cost, even though it might not be necessary to remove the sand to quite as great a depth each time. Dr. Woodward.

After all, the only reason for adding a coagulant seems to be to guard against the appearance of any turbidity in the effluent; and, as a medical man and a sanitarian, the writer does not take the question of turbidity at all seriously. He cannot agree with the authors of the paper that the suspended matter or turbidity is that quality of the water that makes the greatest impression upon the public. It is granted, of course, that the turbidity might be so disproportionately high, in comparison with the morbidity and mortality rates from water-borne diseases, that it might attract more attention than the latter; but, as between a high morbidity and mortality from water-borne diseases and a turbidity merely correspondingly high, the public would be impressed in the long run more strongly by the sickness and death than by the imperfect transparency of the water. As an official of a municipal government that is striving hard to better the physical condition of the people under its care, the writer questions the propriety of the expenditure of public funds solely for the removal of turbidity as long as there are so many other outlets for the money that promise more definite sanitary and social returns. A city without a public bath-house, without a playground worthy of the name, with miles of marshland on its very borders, without a sanitarium for the treatment of indigent consumptives in the early stages of the disease, with alleys crying earnestly to be opened up—such a city, particularly when there is no public complaint of the muddiness of its water supply, is not in a position to expend $100 000 and more to establish a clarification system, and thousands of dollars annually for its maintenance.

The possible effect of the use of aluminum sulphate or other coagulant on public health is not considered herein. Too little is known about public health—and too little about private health,

Dr. Woodward.

either, for that matter—to speak dogmatically concerning it. From the experience of millions of millions of people, extending over thousands of years, it is known, however, that pure water which has not been treated with a coagulant is a safe beverage, and those who seek in any given instance to introduce a coagulant into a public water supply must bear the burden of proof to show that without its use pure water cannot be obtained, and that from its use no harm can come, not even through human frailty, which is not less than the danger that will result solely from the omission of the chemical.

All this is written in no spirit of criticism of the filter or of its work, nor for the purpose of suggesting any impracticable remedies for dangers real or fancied. The writer's purpose has been to point out the fact that there is no substantial evidence that the system of filtration endorsed by the medical profession for use at Washington cannot be made to yield perfectly satisfactory results, from the standpoint of bacterial efficiency and of turbidity. The writer knows of no evidence that indicates even that the very plant that has been constructed has not given satisfactory results when gauged by the standard of bacterial efficiency, and the only evidence that has ever come to his knowledge that it has not yielded an effluent satisfactorily clear has come from those concerned in the operation of the plant, who appear to have been dissatisfied at times because they were unable to attain the high standard of brilliancy in the effluent that they have set before them. The public seems to be satisfied.

If the writer were asked, however, what practical benefit had resulted during the year that the filters have been at work, he would be compelled to say that filtration had done but two things: First, it has given a drinking water which has been clear enough to satisfy any reasonable person; no complaint of turbidity has been heard, there has been no ground for such a complaint. Second, the filtration plant has stood between the people of Washington and the inhabitants of the basin of the Upper Potomac River to safeguard them against possible infection from that source; if, however, the filtration plant has actually prevented a single case of typhoid fever or of other water-borne disease, evidence of it has not been discovered. It is not stated that it has not saved many from such diseases, for it is not known how many might have been afflicted if the filtration plant had not been in operation, but if it has prevented any cases whatsoever the writer has not found any evidence of that fact. The design or operation of the filters are not being blamed; from the bacteriological records relating to the operation of the plant, it would be unsafe for anyone to do so without direct evidence.

The writer is being forced to the conclusion that all were mistaken in blaming the public water supply for the prevalence of typhoid fever in Washington. It may be, however, that the boiling

of drinking water and the use of bottled waters for drinking, which had become rather common practices after the real extent of the prevalence of typhoid became known, had eliminated a considerable part of the water-borne typhoid fever before the filtration plant was put into operation, for about July, 1903, there occurred a sudden drop in the morbidity and mortality rates, and the lower rates then established have continued ever since. In any event, the writer is unable to formulate any hypothesis to explain the present excessive incidence of this disease in the District of Columbia that is consistent with the accepted theory as to the causation of typhoid fever, *viz.*, that typhoid fever is due to the ingestion of certain bacilli which have come through either urine or feces from a patient suffering from that disease. But no efforts have been abandoned for the prevention of this disease along lines already laid down: the closing of public wells as rapidly as circumstances justify, the abolition of box privies, and the improvement of the milk supply; and it is expected that the filtration of the water supply will be continued until the end of time. It is to be hoped that the discussion of this paper may point to some remedy which will afford more speedy relief than is promised by any or all the measures heretofore undertaken and still in progress. Dr. Woodward.

GEORGE M. KOBER,* M. D. (by letter).—The writer has read with much pleasure this excellent paper, and has considered the question which naturally concerns the sanitarian most, *viz.*, the bacterial efficiency of the filtration system and its effect upon lessening the typhoid fever rate; and here one is at once confronted with the remarkable fact that, while the bacterial efficiency of the Washington plant is all that could be desired, the average number of bacteria in the effluent being only 29 as compared with 4 900 in the raw water, no apparent decrease in the number of typhoid fever cases has resulted during the first year of its practical operation. This would seem to indicate that the Potomac water is either not as important a factor in the prevalence of typhoid fever as heretofore assumed, or that seasonal and telluric influences have been exceptionally favorable for a large crop of typhoid this year, and that the situation would have been infinitely worse had it not been for the efficiency of the filtration plant. The latter explanation should be accepted until further observation compels a different view. A decrease of 50% in the typhoid fever rate was confidently expected, as the writer believes that about 50% of the cases are water-borne, and this, too, in spite of the fact that he was the first to point out the agency of flies in the spread of the disease; about 15% of all cases are brought to the city from mountain resorts by returning vacationists; milk produced in typhoid-ridden districts is probably responsible for 15% Dr. Kober.

* Georgetown University, Washington, D. C.

Dr. Kober. of the cases, and the remainder, about 20%, may be spread through the agency of flies, personal contact, the consumption of raw oysters raised in sewage-polluted oyster beds, or the eating of strawberries, radishes, celery, lettuce or other vegetables which have been contaminated by infected night-soil. The writer has not had the time or opportunity to examine into the typhoid situation elsewhere, but has suggested, to those now engaged in the investigation of the subject, the desirability of making such a comparison. He is not disposed to exclude the "drinking-water theory" altogether, or to regard the amount of typhoid fever now remaining as purely "residual," because all the other factors have operated in former years, and there is no reason to assume that the milk supply is any worse now than it was ten years ago, for, as a matter of fact, since the enactment of a pure-milk law in 1895, there has been a decided decrease in the death rate of children under five years of age; indeed, the percentage of deaths of children under one year has been reduced from 26.94 in 1893 to 18.36 in 1905, due solely to the diminution of the number of deaths among children, and not to an increase in the total number of deaths. The gradual reduction in the number of box privies, from 30 000 in 1871 to about 4 000 in 1905, should also have resulted in a reduction of the danger from flies in the transmission of the germs. The same may be said of public wells, which have been reduced from 171 in 1895 to 102 in 1905; the number of private wells, however, is still a matter of conjecture. As long as public wells are permitted to exist at all, it is difficult to estimate the part they have in the dissemination of typhoid fever; they are promiscuously used, and, as they are located in a polluted subsoil, they must be regarded as a constant source of danger, and, upon general principles, should be condemned. The gradual elimination of public wells appears to be an extremely conservative policy, because wells which from time to time are declared infected have passed the scrutiny of the chemist and bacteriologist for years before. The Rivers Pollution Commission,* more than 25 years ago, advised the closing of all the wells in London but three which were exceptionally placed.

To the student of epidemiology, the situation, on the whole, is extremely interesting and perplexing, and indicates that there is yet much to be learned regarding the causes and spread of typhoid fever; but the writer has not ceased to hope that the typhoid situation in Washington will clear up, as it did in Berlin.

In 1872 Berlin and Washington were alike in primitive municipal methods of sanitation. Without adequate distribution of the water supply, the larger part of their population was dependent upon public and private wells. Without sewerage systems, both were

* Sixth Report, p. 284.

alike supplied with makeshifts for the disposal of human excreta and waste waters. The unpaved streets were the common receptacle of garbage, ashes and house sweepings, and along the gutters the surface and house drainage were conducted to the neighboring watercourses. Since Berlin began its system of sanitary improvements, completed in 1890, at a cost of 59 000 000 marks for a pure and ample water supply and 42 500 000 marks for the construction of sewers, the typhoid fever death rate has fallen from 143 per 100 000 of its inhabitants in 1872 to 4 in 1894; and the general death rate has fallen from 32.9 per 1 000 in 1875 to 16.4 per 1 000 in 1904, while the death rate from typhoid fever in Washington is still six times as great as that of Berlin, yet the general death rate has been reduced from 28.08 in 1875 to 19.20 in 1905, which is encouraging, considering the mixed character of the population. Dr. Kober.

In 1875 only 57 houses in Berlin were connected with the sewers, and the general death rate was 32.9 per 1 000. In 1885 there were 15 895 houses with sewer connections, and the death rate was 24.4; the Washington rate was 24.9. In 1895 the Berlin death rate was 19.8, and the Washington death rate was 20.5.

In conclusion, the hope is expressed that the recommendation of the authors for the installation of a coagulating process in periods of great turbidity, may be speedily acted upon, and that the general installation of meters will lessen the waste of water and thus permit of a slower rate of filtration. These recommendations cannot fail to increase the efficiency of the plant.

THEODORE A. LEISEN, M. AM. SOC. C. E. (by letter).—This paper is extremely interesting, and makes a valuable addition to the literature on filtration of public water supplies. The reference to the proposed preliminary treatment of the water opens up a subject not as generally familiar as that of the final or ordinary treatment, by slow sand filtration. This question has occupied a share of the writer's attention for the past few years, in connection with the proposed filtration of the water supply of Wilmington, Del., and, while financial, legal and other complications have been the means of preventing or delaying the actual construction of the contemplated works, the delay has afforded time and opportunity to carry out some work on experimental lines which, in the end, may prove beneficial to the cause. Mr. Leisen.

The principal object of the experimental work was the determination of the utility of preliminary treatment of the water preparatory to final filtration by the slow sand system: First, in establishing its value in increasing the efficiency of the sand filters; and second, in reducing the cost of operation.

The experiments were along mechanical lines, in contradistinction to chemical treatment, where a coagulant is required.

Mr. Leisen. The experimental plant consists of two preliminary filters, each 4 ft. wide by 11 ft. long, having an effective area of $\frac{1}{1000}$ acre, and three slow sand filters, having an area equivalent to $\frac{1}{6000}$ acre. The preliminary filters receive the raw water directly from Brandywine Creek (which is the source of supply for the city). Two of the final sand filters receive the pre-filtered water from the preliminary filters, while the third takes the water directly from the creek. The slow sand filters are constructed in the usual manner, and call for no special comment.

Three types of preliminary filters have been used in the experiments. Filter No. 1, generally known as the Maignen filter, was constructed on lines advocated by J. P. A. Maignen, Assoc. Am. Soc. C. E. It consists of a lower strainer course of large gravel or broken stone, above which is placed about 14 in. of coke, and on top of this, 20 in. of sponges, which are slightly compressed, and held down by a wooden rack. The water is introduced from below, through a perforated pipe, filtering upward, and overflowing at the top.

Filter No. 2 was constructed on lines similar to those of mechanical filters, sand being the filtering medium. A series of pipes with perforated strainers serves to carry off the filtered effluent, and also to introduce air and water for washing purposes.

Filter No. 3 is similar in general design to Filter No. 1, except that charcoal is substituted for coke, and the water is taken in at the top and flows downward, discharging through the strainer system at the bottom.

From the information obtained previous to commencing these experiments, it was believed that the sponge filter (No. 1) would operate for a period of from 6 to 12 months without undue loss of head, or excessive clogging, but the experience of the first few months demonstrated conclusively the falsity of this hypothesis. Within a month after starting, springs formed, materially interfering with the efficiency, and a reversal of the current (the only means then provided for partially washing) did not completely remedy the evil. At the end of two months the loss of head due to clogging was so great that the sponge had to be removed and washed.

In an effort to provide more efficient means for washing this filter, air wash-pipes were introduced early in February, since which time the work of this filter has been uniformly satisfactory. It has been deemed expedient to limit the loss of head to 12 in., and, under the conditions existing since February, during which time there have been periods of excessive turbidity, the periods between washings have averaged about 30 days. The rate of filtration is about 50 000 000 gal. per acre per 24 hours.

Filter No. 2 was operated for several months, but it was found to clog up so rapidly—with a consequent loss of head and decrease in rate—and to require such constant attention and such an excessive quantity of wash water, that it was thought advisable to abandon it, as unsuited to the requirements. Mr. Leisen.

Filter No. 3 was in operation about 5 months, and, while it showed a very high efficiency at the start, with slight loss of head, it was found to be unsatisfactory in the end, as a mud layer several inches thick, which formed on top of the sponges, broke through at intervals and allowed the water to course through, practically nullifying the efficiency. After repeated efforts to overcome this difficulty, without success, the filter was abandoned.

Filter No. 1 has removed as little as 33% of turbidity, and occasionally as much as 100%, but the average since February, when the air-wash was introduced, has been 65% reduction of turbidity, while the average bacterial reduction has been about 65 per cent. For brevity, all detailed statements of bacteria and turbidity are omitted.

The influence of this preliminary treatment on the final slow sand filters may be summarized in the statement that no appreciable difference is found in the filtered effluents from the creek water and from the pre-filtered water, in so far as the final bacterial efficiency is concerned, but the final clarification is improved by pre-filtration. Its influence on the economy of operation, however, is more marked, as the evidence so far obtained has demonstrated that the quantity of water passed through the slow sand filters for each run is doubled by the use of the preliminary filter; or, in other words, in the filters fed with pre-filtered water, the period between scrapings has been twice as long as that between scrapings of those receiving creek water.

The advantage attained in prolonging the duration of runs in the slow sand filters insures both greater economy and increased uniformity, with a consequent increase in general efficiency, for, as every disturbance of the sand beds produces deterioration in the quality of the filtered effluent, so any treatment that minimizes the number of scrapings will evidently tend toward higher average results.

It may be assumed that the preliminary treatment referred to herein is simply a rapid system of sedimentation, and its economic value will depend, first, upon a comparison of the cost of construction and maintenance of a sedimentation basin and a preliminary filter of equal effective capacity; and second, upon the combined cost of construction and maintenance of a slow sand filter with and without the preliminary filters. Both these considerations are questions which have to be determined separately in each individual case.

Mr. Leisen. The data, upon which the foregoing is based, are too meager, and cover too short a period to warrant any conclusive deductions, but are here presented as a subject for discussion, pending further experiments.

Mr. Butcher. W. L. BUTCHER, ASSOC. M. AM. SOC. C. E. (by letter).—Developments in sanitary engineering are taking place so rapidly that a knowledge of the merits and faults of the latest completed works must be considered absolutely necessary to the engineer striving to keep abreast of the times in this branch of the profession.

Perhaps the subject of most immediate usefulness to the designer of water purification plants is that relating to the hydraulics of the sand-handling apparatus. The writer has seen no account of similar experiments conducted elsewhere.

These investigations were a part in the development of one of the most interesting features of the works; namely, the installation for lessening the cost of removing, cleaning and replacing the sand. The authors' comparison of labor expenses at Washington with those at older plants, while instructive, does not show the actual saving in total cost brought about by mechanical means. For such a comparison with older plants, where nearly the whole expense may be charged to the cost of labor, some account must be taken of the capital invested to make the saving possible.

Estimating from the figures in the paper, it would appear that about 20 or 25 cents per cu. yd. should be added for interest on the capital invested. Presumably, also, the wages of the laborers are much less than at the more northern works. The cost of handling sand at Philadelphia is stated to be $1.20 per cu. yd., the latest annual reports of the Board of Public Works being given as the authority. From the 1905 report, the writer computes the average expense as 83 cents per cu. yd.

In the case of such large filters, with their many acres of sand to be scraped at frequent intervals, might it not be economical and practicable to install some sort of mechanical scraper? This might take the form of a light traveling crane operated by an electric motor on rails attached to the piers. An inclined and adjustable scoop about 1 ft. wide would convey the sand to a detachable hopper holding about 1 cu. yd., or the quantity collected in one passage from end to end of the bed. On reaching the end of the filter the hopper would be run directly over an ejector, and be detached and replaced by an empty receptacle for the return trip.

The trend of present practice seems to be away from the use of wholly automatic devices for regulating the flow of the effluent—such as are used at Philadelphia—the opinion being, apparently, that, as the rate of change of loss of head is so slow in the case of slow sand filters, the value of automatic regulating devices is not

commensurate with the difficulties involved in their construction. This, of course, is not the case with mechanical filters. Mr. Butcher.

The results of the first few months of operation have emphasized the value and necessity of investigations previous to construction. Fortunately, in the case of the Washington plant, no considerable outlay is required to bring the works into such shape as to conform more closely with the recommendations derived from the preliminary investigations.

J. P. A. MAIGNEN, ASSOC. AM. SOC. C. E.—In describing the construction of the walls of the pure-water reservoir the authors conclude thus: "A lighter steel-concrete wall, or a buttressed concrete wall, might have been used, with some saving of expense." The writer would like to know why this was not done. Mr. Maignen.

The system of compensating orifices intended to prevent "that part of the filter nearest the outlet to operate at a rate much higher than intended," is commendable. The speaker has used it at Lower Roxborough and elsewhere, and it works well in comparatively small filters, operated at a high and nearly constant rate of filtration; but, in a filter unit with an area of 43 560 sq. ft., with rates of filtration varying from 1 000 000 to 4 000 000 gal. per acre per day, the conditions are different. The orifices made for the 4 000 000-gal. rate are too large for the 1 000 000-gal. rate. What evidence is there that the orifices computed for the high rate will act satisfactorily for the low rate?

The stone and gravel underdrains are 12 in. in thickness—the first layer being 7 in. and the other two 2½ in. each. This comparatively thin layer of gravel might be sufficient for small filters, carefully handled; but, for units of 1 acre the speaker would prefer deep layers of underdrain materials.

The quality of the sand used appears to be satisfactory. It is of small size, as compared with the Philadelphia sand. The method used for preparing it at the bank is quite interesting. Nothing that can be done for producing a good filter sand is to be accounted as extravagant. If economies are to be made, they are not to be tried on the materials that do the work. They are to be sought in the general design of the plant and in the accessories.

The water is pumped directly from the Washington City Reservoir to the filters, without passing through any compensating reservoir. The authors, in accounting for this omission, say "there is no space available * * * on the site selected, all of which is required for the filters." If a compensating reservoir with a capacity of 3 000 000 or 4 000 000 gal. had been really thought necessary, space could easily have been found. The sand courts, for instance, could have been reduced in size; or preliminary filters could have been installed, with the object of increasing the rate of final filtration and

Mr. Maignen. reducing the number of final filters. This direct pumping appears to be a departure from what is understood as the best practice.

The authors say:

"The pure-water reservoir is arranged so that when it becomes full it will back up water against the filters and reduce the rate of filtration automatically."

Is this right?

The general practice in operating slow sand filters is to disturb the filtering medium as little as possible. So-called automatic mechanical devices of every description have been designed with a view of insuring a constant rate of filtration, but, in general, hand-regulation has been found the safest. The filter attendant, at first, keeps the outlet valves partly closed, and then, as the friction in the upper portion of the sand bed increases, which phenomenon is indicated by the loss of head, he opens the valve little by little, day after day, endeavoring all the time to maintain the rate of filtration as constant as possible so as not to disturb the filtering material. An inattentive filter attendant who opens the valves too much or too little in the middle of a run, and thus brings about a variation in the rate of filtration, will not retain his position very long. The automatic reduction and acceleration of the rate of filtration, as the water rises and falls in the Washington pure-water basin, commended by the authors, does, every day, and several times a day, precisely what the filter attendant is instructed not to do.

The outlet valves which lead the filtered water into the pure-water reservoir are placed at an elevation of about 2 ft. from the bottom of the filters (the top of the 20-in. outlet valve is 2 ft. 8 in. above). This is a feature which is somewhat unusual, and does not appear to be desirable. Taken in combination with the backing up of the water from the reservoir, this feature must have a tendency to keep the sand layer, below and above the outlet valve, in a loose condition. This loose state of the sand may perhaps account for the "abnormally long period between scrapings," and the soiling of the sand "to a most unusual depth." The authors think that this penetration is due to the fact that the water is so well settled before it reaches the filter that it "does not contain particles of a size to form at the surface of the sand the sediment layer usually obtained in the operation of sand filters." This defect, if it should really exist, could be easily overcome by putting in the water those particles which are said to be wanted; these can be procured without coagulation.

The authors describe at some length the sand-washing process, and the illustrations show the very picturesque sand bins. These are certainly an improvement on the old sand courts, but the speaker thinks that the bins are as unnecessary as the courts; both are likely

to be ignored in future filter plants in favor of sand-washing systems, in or about the filter bed, with the immediate return of the sand to its working place. The speaker does not like to see dirty water going into the bin with the sand. He also objects altogether to the ejector system, mainly because it wastes or separates the finer from the coarser particles of sand, and thus gradually reduces the efficiency of the whole. Mr. Maignen.

The authors do not say anything about preliminary filtration. They insist on coagulation. The speaker maintains that, as the average turbidity of the water in the Washington City Reservoir does not exceed 30 parts per million, it is possible to deal with it by physical methods. Coagulation was strongly objected to, for hygienic reasons, when the matter came up originally before Congress. The speaker now objects to it for financial reasons. The authors say it will cost 50 cents per million gallons for all the water treated throughout the year. Preliminary filtration, with such modifications in the final filters and their operation as might be found necessary, would not add anything to the total cost of filtration. The cost of the installation and operation of preliminary filters would be more than compensated by economies in the operation of the final sand filters and by the higher efficiency obtained.

TABLE 11.—Typhoid and Enteric Fever at South Bethlehem, Pa. Number of Cases and Deaths.

Month.	1903.		1904.		1905.		1906.	
	Cases.	Deaths.	Cases.	Deaths.	Cases.	Deaths.	Cases.	Deaths.
January	17	4	5	5	1	0	1	0
February	6	14	6	5	1	0	1	0
March	68	7	?	5	0	0	0	0
April	66	12	8	2	0	0	4	0
Totals	157	37	21	17	2	0	6	0

Totals for 1904 and 1905, before the filter plant was installed.*		Totals for 1905 and 1906, after the filter plant was installed.	
Cases.	Deaths.	Cases.	Deaths.
178	54	8	0

* The filter plant was placed in operation in November, 1904.

As allusion has been made to the prevalence of typhoid fever at Washington, it may be of interest to know the influence which filtra-

Mr. Maignen. tion has had on this disease at South Bethlehem. Table 11 is the month record of typhoid and enteric fever for the four months of the year in which it is most prevalent—January, February, March, and April—during the last four years.

Mr. Henny. D. C. HENNY, M. AM. SOC. C. E. (by letter).—The writer visited the Washington filtration plant, both before and after its completion. He believed that pure utilitarianism had been carried to its extreme in the western part of the United States, on works built for quick profits and then only in remote localities, but confesses that, when visiting the completed Washington filtration plant, he was struck by the total absence of any attempt at architectural treatment, except as to the pumping station and the depressed entrances to the filters.

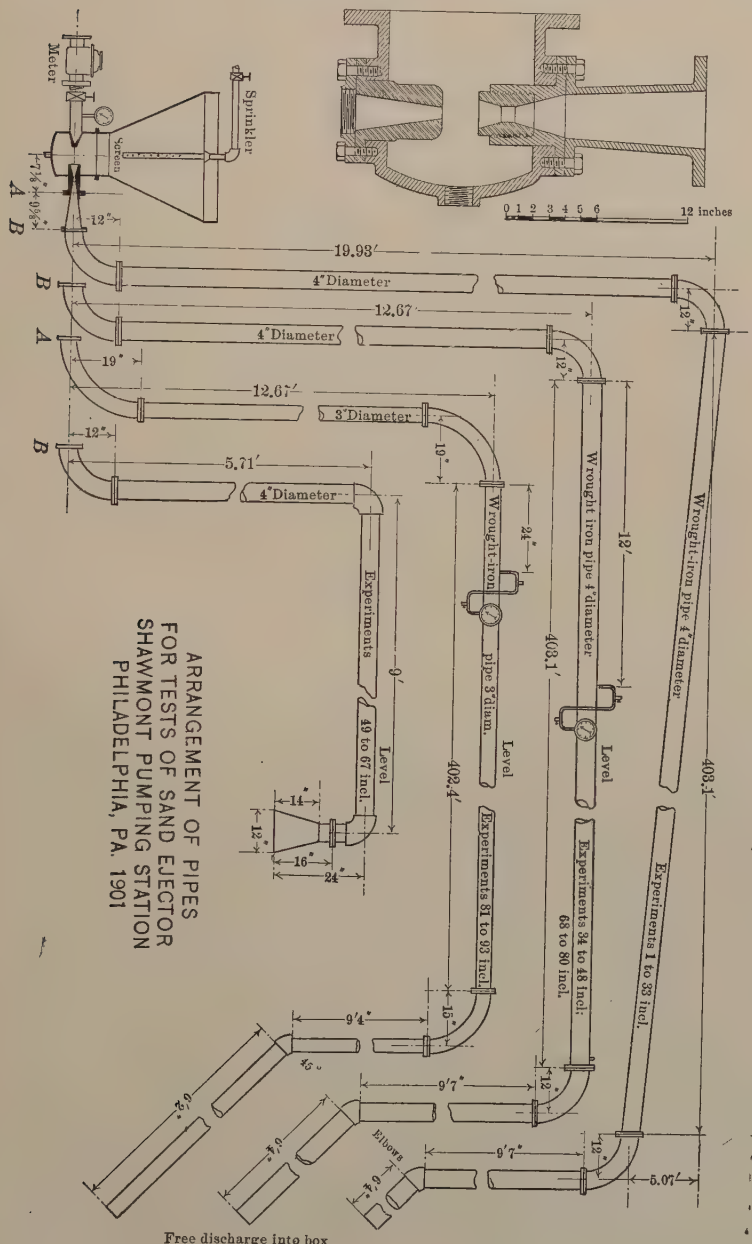
By far the most conspicuous parts of the plant are the elevated, concrete sand bins. Whether these could have been built in a depressed position and the washed sand could have been returned to the filters by utilizing the pumping plant and pipe lines already provided, is a matter which the authors have undoubtedly considered, but possibly only from an economic standpoint. As built, these sand bins are elevated, and, aided by their great number, they attract attention from afar.

Large smooth surfaces of concrete are never attractive in appearance, being unrelieved by joints and often discolored in unsightly ways. There are fine examples, however, of simple, artistic and comparatively cheap treatment of concrete surfaces. In this instance, no attempt appears to have been made to relieve the barren monotony.

Much may be written about the justification of the expenditure of public money for architectural effect, in connection with works of public necessity, and differences of opinion in regard thereto will always exist. If such expenditure is justified anywhere, however, it is believed to be in this instance, on prominent monumental structures built by the United States Government within the confines of the City of Washington.

Mr. Vogleson. J. A. VOGLESON, ASSOC. M. AM. SOC. C. E. (by letter).—The authors have called attention to the increase in the cost of filter construction at the present time over that of earlier works of the same character. They also show marked reductions in the cost of some of the items incident to the operation of slow sand filters. That the cost of operating is one of the principal points at which to attack filtration problems at present is made of greater importance by the increased costs of construction of which the authors speak. At the slow sand plants now in operation, or in process of construction, the handling of large quantities of sand each year

Mr. Vogleson.



ARRANGEMENT OF PIPES
FOR TESTS OF SAND EJECTOR
SHAWMONT PUMPING STATION
PHILADELPHIA, PA. 1901

FIG. 19.

Mr. Vogleson.

is a considerable factor in the cost of operation, and data on the sand problem are of no little importance.

In 1901, the writer, while engaged with the Bureau of Filtration at Philadelphia, under the direction of John W. Hill, M. Am. Soc. C. E., was assigned to conduct the tests of a sand ejector and washer built for experimental purposes. Those tests demonstrated the feasibility of an ejector system, but time did not permit a complete solution of many of the interesting problems which developed during the course of the experiments, and the data here presented are given with the hope that they will assist others who have to deal with sand transportation.

The experimental plant was installed at the Shawmont Pumping Station, where water under the desired pressures, and other necessities were available. In general, it consisted of a supply line on which a meter and a pressure gauge were located; a cast-iron feed hopper, with a base constructed in such a manner that a horizontal jet of water crossed near the bottom from a nozzle to a throat; a delivery pipe which was varied in diameter, length, and grade, as shown on Fig. 19; a perforated pipe, called the sprinkler, placed in the vertical axis of the hopper for the purpose of washing the sand through the screen near the base of the hopper; an assortment of nozzles and throats; and a supply of sand, with bins for storing and measuring it, and the necessary implements for handling it.

The method of conducting a test was as follows: Having placed a particular arrangement of nozzle and throat in the ejector, and having regulated the water pressure, a measured quantity of sand was wheeled to the hopper in barrows and dumped as rapidly as the ejector would carry it away. Each combination was worked to its full capacity, and check observations, for time, and quantities of sand and water, were made at frequent intervals by barrow count and meter readings. The sand was caught in a bin placed at the lower end of the discharge pipe, and carted back to the upper end. Thus the same sand was used repeatedly, and, as shown in Table 17, the change in effective size due to repeated handlings was noted, as the idea had been advanced that a large quantity of the smaller particles might be carried away in the overflow.

The first series of experiments, Table 12, was made to cover varying pressures, and various diameters of nozzles. The ejector was fitted with a $1\frac{7}{16}$-in. throat which wore down to $1\frac{3}{4}$ in. during the tests. The diameter of nozzle was varied from $\frac{3}{4}$ in. to 1 in. The water pressures ranged from 30 to 80 lb. per sq. in. The delivery pipe was a 4-in. wrought-iron pipe, and was laid on a trestle, so that the vertical lift from the axis of the jet to the head of the long section of the delivery pipe was 19.93 ft., and there was a fall of 5.07 ft. in the length of 403.1 ft. At the end of the long section the

DIAGRAMS SHOWING THE RESULTS OBTAINED BY VARYING THE SIZE OF THE NOZZLE, AND THE WATER PRESSURE AT THE NOZZLE, FOR EJECTING 0.26-M.M. SAND THROUGH 403.1 FT. OF 4 IN. PIPE WITH 5.07 FT. FALL. THROAT 1 7/16 IN. IN DIAMETER WEARS TO 1 3/4 IN. DURING EXPERIMENTS. DISTANCE FROM NOZZLE TO THROAT 2 3/4 IN. TO 3 IN.

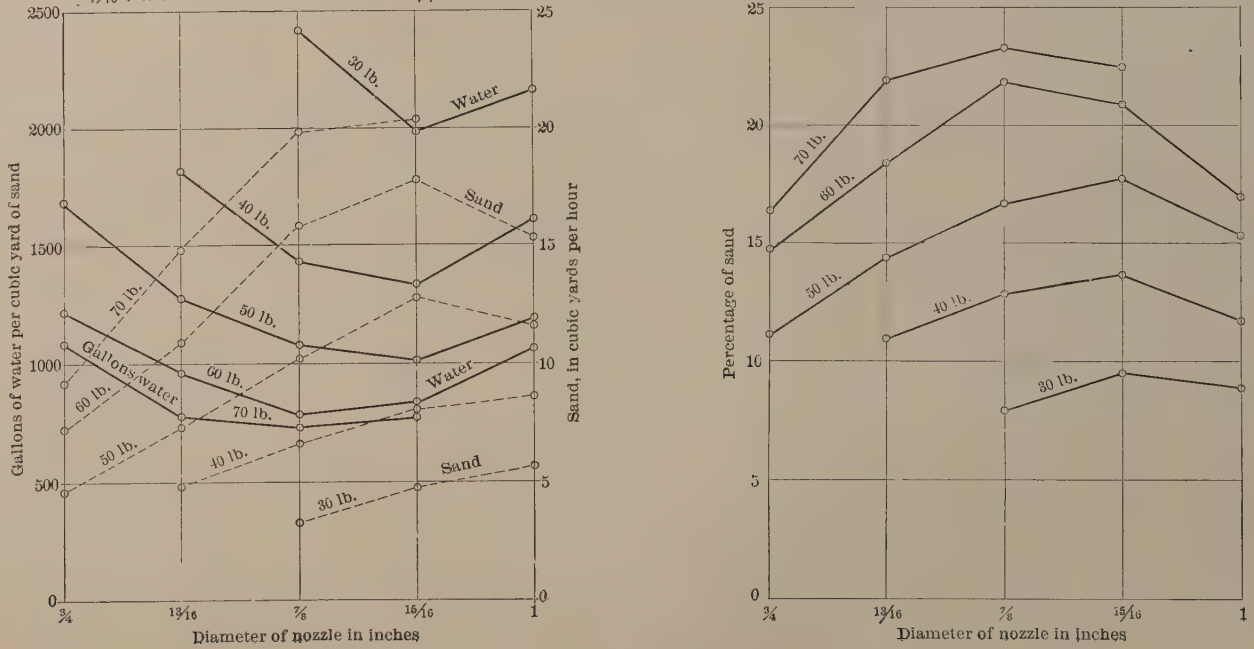


FIG. 20.

Mr. Vogleson.

grade changed abruptly in order to discharge the mixture into the catch-box. Bar sand of an effective size of 0.26 mm. was used. The results, plotted on Fig. 20, show that the maximum percentage of sand in the mixture, for pressures of 30, 40 and 50 lb. per sq. in., was obtained with a $\frac{5}{16}$-in. nozzle for the size of throat and the distance from nozzle to throat as given, and for pressures of 60 and 70 lb. per sq. in., the maximum percentage was reached with a $\frac{7}{8}$-in. nozzle. At this stage of the experiments, instructions were received

TABLE 12.—TESTS OF SAND EJECTOR AT SHAWMONT PUMPING STATION, PHILADELPHIA, PA., 1901.

Ejecting sand through 403.1 ft. of 4-in. pipe. Lift = 19.93 ft.

Delivery pipe laid with a fall of 5.07 ft. in 403.1 ft.

Experiment Number.	Diameter, in Inches.		Distance from nozzle to throat, in inches.	Water.				Sand.			Mixture.		
	Nozzle.	Throat.		Pressure in supply pipe, in pounds per square inch.	Gallons used per cubic yard of sand transported.	Cubic feet per minute.	Ratio of water used to sand transported.	Cubic feet of sand transported per minute.	Cubic yards of sand transported per hour.	Effective size of sand, in millimeters.	Cubic feet of mixture transported per second.	Velocity, in feet per second.	Percentage of sand in mixture.
5	1	1 7/16	2 3/4	40	1 959	28.94	9.78	2.98	6.60	0.26	0.501	5.74	9.6
6	1	1 7/16	2 3/4	40	1 606	30.24	7.95	3.83	8.51	0.26	0.547	6.26	11.7
7	1	1 7/16	2 3/4	50	1 180	31.90	5.84	5.46	12.16	0.26	0.592	6.78	15.3
8	1	1 7/16	2 3/4	30	2 162	25 71	10.70	2.40	5.33	0.26	0.455	5.21	8.8
9	3/4	1 7/16	2 3/4	70	1 093	22.51	5.42	4.16	9.23	0.26	0.421	4.82	16.4
10	3/4	1 7/16	2 3/4	60	1 218	19.87	6.03	3.29	7.32	0.26	0.368	4.21	14.9
11	3/4	1 7/16	2 3/4	50	1 680	17.62	8.32	2.11	4.68	0.26	0.317	3.63	11.1
12	3/4	1 7/16	2 3/4	80	774	23.41	3.84	6.11	13.58	0.26	0.458	5.24	22.2
13	3/4	1	2 3/4	70	1 226	21.35	6.08	3.51	7.81	0.26	0.395	4.56	14.8
14	3/4	1	2 3/4	80	947	23.45	4.68	5.02	11.15	0 26	0.447	5.14	18.7
15	1	1 7/16	2 3/4	60	1 058	35.88	5.23	6.86	15.25	0.26	0.674	7.72	16.9
16	1 3/16	1 7/16	2 7/8	50	1 268	21.03	6.28	3.36	7.47	0.26	0.388	4.44	14.4
17	1 3/16	1 7/16	2 7/8	60	963	23.54	4.77	4.94	10.98	0.26	0.447	5.12	18.4
18	1 3/16	1 7/16	2 7/8	70	786	26.02	3.89	6.70	14.89	0.26	0.508	5.82	21.9
19	1 3/16	1 7/16	2 7/8	40	1 710	17.97	8.47	2.12	4.72	0.26	0.323	3.70	10.9
20	1 7/8	1 7/16	3	40	1 440	21.74	7.15	3.04	6.75	0.26	0.396	4.57	12.8
21	1 7/8	1 7/16	3	50	1 085	24.50	5.37	4.56	10.13	0.26	0.459	5.26	16.6
22	1 7/8	1 7/16	3	30	2 402	18.18	11.90	1.52	3.38	0.26	0.320	3.66	7.9
23	1 7/8	1 7/16	3	60	789	27.59	3.91	7.06	15.69	0.26	0.538	6.16	21.9
24	1 7/8	1 7/16	2	50	1 060	24.59	5.25	4.68	10.41	0.26	0.462	5.29	16.9
25	1 7/8	1 7/16	2	40	1 420	21.84	7.02	3.12	6.94	0.26	0.397	4.58	13.0
26	1 7/8	1 7/16	4	50	1 142	24.47	5.66	4.33	9.65	0.26	0.456	5.22	15.7
27	1 7/8	1 7/16	4	60	868	27.71	4.29	6.46	14.36	0.26	0.534	6.11	20.2
28	1 15/16	1 7/16	3	30	1 985	20.95	9.84	2.13	4.75	0.26	0.373	4.27	9.5
29	1 15/16	1 7/16	3	40	1 339	24.58	6.63	3.71	8.26	0.26	0.451	5.16	13.7
30	1 15/16	1 7/16	3	50	1 005	28.58	4.97	5.74	12.79	0.26	0.540	6.18	17.7
31	1 15/16	1 7/16	3	60	835	32.95	4.13	7.97	17.70	0.26	0.637	7.29	20.8
32	1 15/16	1 7/16	3	70	763	35.00	3.77	9.26	20.60	0.26	0.686	7.85	22.5
33	1 7/8	1 7/16	3	70	729	32.40	3.61	8.98	19.98	0.26	0.640	7.73	23.3

TABLE 13.—TESTS OF SAND EJECTOR AT SHAWMONT PUMPING STATION, PHILADELPHIA, PA., 1901.

Ejecting sand through 403.1 ft. of 4-in. pipe. Lift = 12.67 ft. Delivery pipe level.

Experiment number.	Diameter, in inches.		Distance from nozzle to throat, in inches.	Water.				Sand.			Mixture.			S.	c, in the Chezy formula, $v = c\sqrt{rs}$.
	Nozzle.	Throat.		Pressure in supply pipe, in pounds per square inch.	Gallons used per cubic yard of sand transported.	Cubic feet per minute.	Ratio of water used to sand transported.	Cubic feet of sand transported per minute.	Cubic yards of sand transported per hour.	Effective size of sand, in millimeters.	Cubic feet of mixture transported per second.	Velocity, in feet per second.	Percentage of sand in mixture.		
34	3/4	1	2 3/4	70	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
34a	3/4	1	2 3/4	70	3 129	20.72	15.56	1.33	2.95	0.26	0.360	4.12	6.1	0.0354	76
35	3/4	1 1/4	2 3/4	70	1 329	21.66	6.56	3.29	7.31	0.26	0.396	4.55	13.8	0.0589	65
36	3/4	1 1/2	2 3/4	70	1 082	21.53	5.56	4.03	8.95	0.26	0.405	4.64	16.6	0.0648	63
37	3/4	1 3/4	2 3/4	70	1 207	21.67	5.96	3.63	8.07	0.26	0.400	4.58	15.1	0.0648	63
38	3/4	2	2 3/4	70	1 667	20.35	8.25	2.46	5.47	0.26	0.366	4.19	11.2	0.0530	63
39	3/4	1 7/16	2 3/4	70	1 040	21.34	5.14	4.14	9.20	0.26	0.401	4.59	17.1	0.0589	65
40	3/4	1 5/8	2 3/4	70	1 103	21.20	5.46	3.88	8.62	0.26	0.396	4.53	16.3	0.0589	64
41	3/4	1 5/8	2 3/4	70	1 130	20.90	5.60	3.73	8.30	0.26	0.389	4.45	15.9	0.0530	67
42	3/4	1 3/4	2 3/4	70	1 010	21.37	5.01	4.27	9.48	0.26	0.404	4.63	17.6	0.0530	69
43	3/4	1 3/4	2 3/4	70	1 110	20.99	5.48	3.81	8.47	0.26	0.391	4.48	16.2	0.0530	67
44	3/4	1 3/4	1 3/4	70	965	21.77	4.78	4.55	10.10	0.26	0.412	4.72	18.3	0.0589	67
45	3/4	1 3/4	3 1/4	70	1 311	21.20	6.48	3.27	7.27	0.26	0.389	4.46	14.0	0.0530	67
46	3/4	1 1/2	3 1/4	70	1 242	21.47	6.15	3.49	7.75	0.26	0.397	4.54	14.6	0.0530	68
47	3/4	1 1/2	1 3/4	70	900	21.91	4.44	4.93	10.95	0.26	0.420	4.81	19.6	0.0589	68
48	1 1/16	1 1/2	1 3/4	60	1 180	23.10	5.85	3.95	8.78	0.26	0.429	4.91	16.1	0.0589	70

Washing sand in one hopper. Lift = 5.71 ft. Delivery pipe 4 in. in diameter, 9 ft. long. Level.

49	3/4	1	2 3/4	70	261	4.85	1.29	3.75	8.33	0.26	0.122	1.40	51.0	.....	.....
50	3/4	1	1 3/4	70	290	4.85	1.44	3.37	7.49	0.26	0.118	1.35	47.4	.....	.....
51	9/16	1	1 3/4	60	430	9.44	2.13	4.43	9.84	0.26	0.206	2.36	35.7	.....	.....
52	9/16	1	1 3/4	70	390	10.03	1.93	5.19	11.53	0.26	0.225	2.58	38.4	.....	.....
53	9/16	1	2 3/4	70	375	10.03	1.89	5.40	12.00	0.26	0.227	2.60	39.6	.....	.....
54	9/16	1	2 3/4	60	388	9.44	1.92	4.90	10.89	0.26	0.211	2 42	38.6	.....	.....
55	9/16	1	2 3/4	50	386	8.31	1.91	4.35	9.67	0.26	0.187	2.14	38.8	.....	.....
56	9/16	1	3 3/4	60	402	9.44	1.99	4.74	10.53	0.26	0.210	2.41	37.6	.....	.....
57	9/16	1	3 3/4	70	390	10.03	1.93	5.19	11.53	0.26	0.225	2.58	38.4	.....	.....
58	3/4	1	3 3/4	70	245	4.85	1.21	4.00	8.90	0.26	0.125	1.43	53.2	.....	.....
59	3/4	7/8	3 3/4	70	622	4.85	3.07	1.58	3.51	0.26	0.099	1.13	26.8	.....	.....
60	3/4	7/8	2 3/4	70	472	4.85	2.33	2.08	4.62	0.26	0.104	1.19	33.3	.....	.....
61	3/4	7/8	1 3/4	70	363	4.85	1.80	2.70	6.00	0.26	0.111	1.27	40.6	.....	.....
62	7/16	1	1 3/4	70	330	6.67	1.63	4.09	9.09	0.26	0.156	1.79	43.5	.....	.....
63	7/16	1 1/8	1 3/4	70	270	6.67	1.33	5.00	11.11	0.26	0.166	1.90	50.0	.....	.....
64	7/16	1 1/8	1 3/4	60	262	5.93	1.29	4.57	10.16	0.26	0.149	1.71	50.9	.....	.....
65	7/16	1 1/8	2 3/4	70	260	6.67	1.28	5.19	11.53	0.26	0.169	1.94	51.2	.....	.....
66	7/16	1 1/8	2 3/4	60	254	5.93	1.25	4.74	10.53	0.26	0.151	1.73	52.1	.....	.....
67	7/16	1 1/4	2 3/4	70	275	6.67	1.36	4.91	10.91	0.26	0.165	1.89	49.4	.....	.....

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that 70 lb. per sq. in. was to be the working pressure, and the combination which would transport 10 cu. yd. of sand per hour with the least quantity of water, should be determined. The proposed installation in covered filters required horizontal pipes. Accordingly, the long pipe on the trestle was made level, so that the lift became 12.67 ft., other conditions remaining practically the same as in the first series of tests. A pressure gauge was placed near the head of the delivery pipe, as shown on Fig. 19, and a piezometer tube was attached at the lower end of the discharge pipe. No measurable head was found at the latter point, but pressures were observed at the

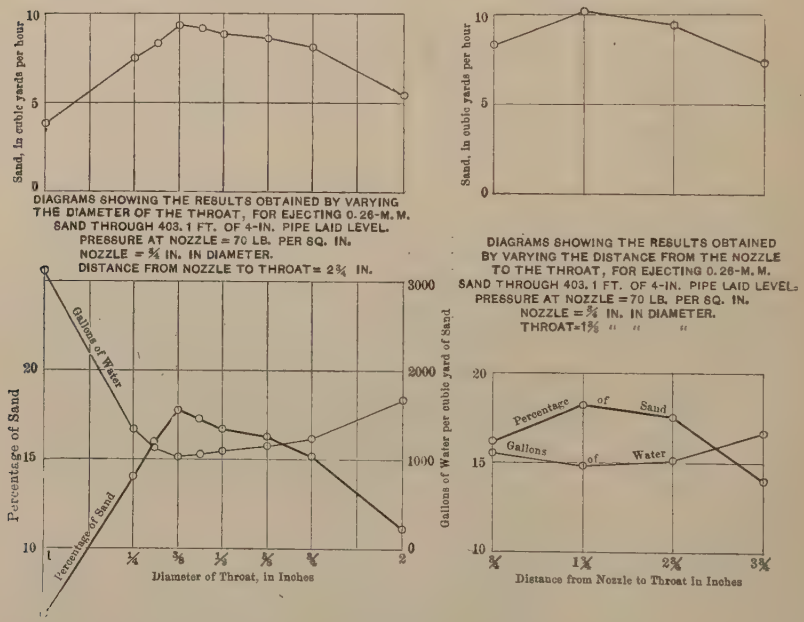
Sand, in cubic yards per hour
10
5
0
DIAGRAMS SHOWING THE RESULTS OBTAINED BY VARYING THE DIAMETER OF THE THROAT, FOR EJECTING 0.26-M. M. SAND THROUGH 403.1 FT. OF 4-IN. PIPE LAID LEVEL.
PRESSURE AT NOZZLE = 70 LB. PER SQ. IN.
NOZZLE = ¾ IN. IN DIAMETER.
DISTANCE FROM NOZZLE TO THROAT = 2¾ IN.
Percentage of Sand
Gallons of Water per cubic yard of Sand
Gallons of Water
Percentage of Sand
3000
2000
1000
0
20
15
10
Diameter of Throat, in Inches
DIAGRAMS SHOWING THE RESULTS OBTAINED BY VARYING THE DISTANCE FROM THE NOZZLE TO THE THROAT, FOR EJECTING 0.26-M. M. SAND THROUGH 403.1 FT. OF 4-IN. PIPE LAID LEVEL.
PRESSURE AT NOZZLE = 70 LB. PER SQ. IN.
NOZZLE = ¾ IN. IN DIAMETER.
THROAT = 1⅜ " " "
Percentage of Sand
Gallons of Water
Distance from Nozzle to Throat in Inches

FIG. 21.

gauge at the head of the pipe from which the slopes of the hydraulic grade were computed. The ¾-in. nozzle was selected for the second series of tests, and the best size of throat was determined; also, the best distance from nozzle to throat. The results are shown in Table 13 and by the diagrams on Fig. 21, and indicate that a 1⅜-in. throat at a distance of 1¾ in. from the nozzle would give the maximum percentage of sand in the mixture, for the rate of 10 cu. yd. of sand per hour, with a water pressure of 70 lb. per sq. in.; and that the delivery could be made through 403.1 ft. of pipe by the use of approximately 1 000 gal. of water per cubic yard of sand. It is also shown that the 1⅜-in. throat can wear to 1¾ in. in diameter before the efficiency of

the ejector is seriously impaired. The adjustments at this stage were considered to have reached practical limits for the ejector, and, comparing Tests 9 and 39, in which all factors are common, except as follows: Mr. Vogleson.

Experiment No.	Net lift, in feet.	Fall, in feet.	Sand, in cubic yards per hour.	Water, in gallons per cubic yard.	Percentage of sand.
9	14.86	5.07	9.23	1 093	16.4
39	12.67	0.0	9.20	1 040	17.1

it was demonstrated that there would be no difficulty with the pipes laid level.

The ejector type of washer was tried in the third series of tests (Experiments 49 to 67, inclusive), shown also by Table 13, and it was found that a $\frac{7}{16}$-in. nozzle at a distance of $2\frac{3}{4}$ in. would lift sand, of 0.26 mm. effective size, 5.71 ft. and deliver it through 9 ft. of horizontal pipe at the rate of 10.53 cu. yd. per hour, using 254 gal. of water per cubic yard of sand per hopper, giving a mixture of 52.1% sand, the nozzle pressure being at 60 lb. per sq. in.

Fortunately, at this time, dirty filter sand was sent to Shawmont, from the Spring Garden Testing Station, in order to learn how many hoppers it would be necessary to place in series to comprise a washer. The special tests of October 9th and 15th, shown on Tables 14 and 15, followed.

The test of October 15th introduced a new factor to be considered—the effective size of the sand. It is to be especially noted in the tests of October 15th that the adjustments were such that a rate of about 9.5 cu. yd. of sand per hour with about 1 000 gal. of water per cubic yard of sand was expected. Only 4.5 cu. yd. per hour were transported, and the water consumption was 2 380 gal. per cu. yd. of sand. Experiments 68 to 80, inclusive, Table 16, were experiments of trial, retrial and verification. It was finally concluded that higher velocities were required. The problem was then how to get the higher velocity without increasing the nozzle pressure and the water consumption. Upon plugging the discharge end of the 4-in. pipe, it was found that a pressure of 35 lb. per sq. in. could be created at the head of the pipe by a $\frac{3}{4}$-in. nozzle, with the water pressure at 70 lb. for the conditions existing in the arrangement. It was considered that the head corresponding to 35 lb. was more than sufficient to force the mixture through 400 ft. of 3-in. pipe. Sufficient confidence was felt in the matter to make a trial of a 3-in. line, and, accordingly, it was erected. Experiments 81 to 93, shown on Table 16, followed, and demonstrated the practicability of the 3-in. line within the prescribed limits; and the piping for the ejector system was designed on that basis. Experiment 90*a* shows that the 3-in. pipe is the best size for the conditions experimented with, for the 0.26-mm. sand as well as the 0.42-mm. size.

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TABLE 14.—ANALYSIS OF SAND AND WATER.
SPECIAL SAND-WASHER TEST, OCTOBER 9TH, 1901.

WASHER:—$\frac{7}{16}$-in. Nozzle. $1\frac{1}{8}$-in. Throat. $2\frac{3}{4}$-in. Distance. 60 lb. Pressure. Water Used: Nozzle, 44.4 + Sprinkler, 15 = 59.4 gal. per min.

SAND:—(Birmingham, N. J., and Edison Mines, Mixed), 1 cu. yd. washed at the rate of 10.5 cu. yd. per hour.

Times washed.	Effective size, in millimeters.	U. C.	60 % finer than.	PERCENTAGES BY WEIGHT PASSING SIEVES, HAVING THE FOLLOWING SEPARATIONS, IN MILLIMETERS.										*Bacteria per gram.
			mm.	0.115	0.160	0.195	0.225	0.30	0.37	0.46	0.57	0.93	1.45	5.00	
0	0.40	1.9	0.75	0.1	0.34	0.81	1.41	4.90	7.97	15.71	31.0	82.1	98.7	100	1 200 000
0	0.40	1.9	0.74	.....	0.35	0.74	1.46	4.93	8.03	16.00	31.5	84.4	98.5	100	.....
1	0.35	2.0	0.70	.....	0.58	1.28	2.28	7.18	11.65	21.86	38.8	87.4	99.0	100	250 000
2	0.35	2.0	0.71	.....	0.54	1.22	2.25	6.97	11.43	21.05	37.4	86.3	98.7	100	110 000
3	0.35	2.6	0.71	.....	0.44	1.08	2.16	6.77	11.05	20.30	37.2	86.6	99.0	99.9	22 000
4	0.35	2.0	0.70	.....	0.44	1.14	2.22	7.30	11.72	21.60	38.8	87.4	99.0	100	11 000
5	0.33	2.1	0.68	.....	0.48	1.22	2.50	7.99	13.48	24.80	43.3	90.6	99.5	100	5 600

WATER:—(Schuylkill).

	PARTS PER MILLION.					
	Raw wash-water.	Overflow from hopper. Times washed.				
		1	2	3	4	5
Silica turbidity	100	45 000	20 000	7 500	3 500	2 000
Total solids	160	24 000	.....	.....	.....	1 400
	*Bacteria per cubic centimeter.					
Average	23 000†	800 000†	200 000‡	65 000†	43 000†	40 000†
Maximum	25 000	1 100 000	240 000	95 000	52 000	46 000
Minimum	20 000	550 000	125 000	40 000	34 000	30 000

*Bacterial counts after 60 hours' incubation.
†Average of 6 samples.
‡Average of 5 samples.

In the experiments with long lengths of 4-in. delivery pipe, velocities of from 4 to 7 ft. per sec. were obtained. The discharge for velocities under 6 ft. per sec. indicated that the sand was but partially suspended and the mixture not complete. The highest percentage of sand attained in the mixture was 19.6 per cent. In the experiments with the washer and the short 4-in. delivery pipe, a mixture containing 52.1% of sand was secured, and flowed like thin

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TABLE 15.—ANALYSIS OF SAND AND WATER. SPECIAL EJECTOR AND WASHER TEST, OCTOBER 15TH, 1901.

EJECTOR: $\frac{3}{4}$-in. Nozzle. $1\frac{3}{8}$-in. Throat. $2\frac{3}{4}$-in. Distance. 70-lb. Pressure. 403.1 ft. of 4-in. pipe. Level. 2 380 gal. of water per cubic yard of sand; 4.5 cu yd. of sand per hour.

WASHER: $\frac{7}{16}$-in. Nozzle. $1\frac{1}{8}$-in. Throat. $2\frac{3}{4}$-in. Distance. 60-lb. Pressure. Water used: Nozzle, 44.4 + Sprinkler, 15 = 59.4 gal. per min. 10.5 cu. yd. of sand per hour.

SAND:—(Birmingham, N. J., and Edison Mines, Mixed). Quantity handled = 28.5 cu. ft.

Condition.	Quantity sampled, in grams.	Effective size, in millimeters.	U. C.	60% finer than: mm.	Percentages by Weight Passing Sieves Having the Following Separations, in Millimeters. 0.115	0.160	0.195	0.225	0.27	0.37	0.46	0.57	0.93	1.45	Bacteria* per gram.
Dirty	383.4	0.42	1.8	0.77	.....	0.26	0.55	1.02	2.11	6.34	13.40	26.8	87.8	98.9	280 000
Ejected	395.5	0.40	1.8	0.72	.....	0.20	0.43	0.81	2.18	7.81	17.48	35.4	87.7	99.1	36 000†
Washed once	387.7	0.40	1.8	0.72	.....	0.26	0.55	0.94	2.33	7.62	16.60	34.2	88.1	99.1	25 000
Washed twice	371.0	0.40	1.8	0.73	.....	0.30	0.59	0.94	2.21	7.01	15.40	32.1	86.2	98.7	16 000
Washed three times	401.4	0.40	1.8	0.72	.....	0.25	0.52	0.97	2.29	7.57	16.90	34.8	88.4	99.0	3 200
Washed four times	387.0	0.37	1.8	0.67	.....	0.28	0.67	1.24	3.04	10.26	21.80	43.9	92.9	99.5	2 500
¶Waste	293.2	0.16	2.2	0.36	5.75	10.15	17.00	23.90	39.40	62.2	75.00	.....	.....	.....	.....

WATER:—(Schuylkill).

	Raw Wash-Water.	Overflow from Hopper. Ejecting.	Times Washing. 1	2	3	4
Silica Turbidity—parts per million	22‡	27 000‡	6 000§	3 600§	2 300§	1 600§
Total Solids " "	180	4 000	3 150	1 900	1 800	1 200
Average Bacteria* per cubic centimeter	8 200	130 000	34 000	20 000	11 000	6 200
Maximum " " " "	33 000	350 000	50 000	28 000	22 000	11 000
Minimum " " " "	900	50 000	6 000	11 000	5 000	3 000

* Bacterial counts after 48 hours' incubation.
† Average of two.
‡ Average of eight.
§ Average of six.
¶ Solid matter overflowing from catch-box in which ejected sand was caught.

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TABLE 16.—TESTS OF SAND EJECTOR AT SHAWMONT PUMPING STATION, PHILADELPHIA, PA., 1901.

Ejecting sand through 403.1 ft. of 4-in. pipe. Lift = 12.67 ft. Delivery pipe level.

Experiment number.	Diameter, in inches. Nozzle.	Diameter, in inches. Throat.	Distance from nozzle to throat, in inches.	Water. Pressure in supply pipe, in pounds per square inch.	Water. Gallons used per cubic yard of sand transported.	Water. Cubic feet per minute.	Water. Ratio of water used to sand transported.	Sand. Cubic feet of sand transported per minute.	Sand. Cubic yards of sand transported per hour.	Sand. Effective size of sand, in millimeters.	Mixture. Cubic feet of mixture transported per second.	Mixture. Velocity, in feet per second.	Mixture. Percentage of sand in mixture.	S.	c, in the Chezy formula, $v = c\sqrt{rs}$.	
68	$\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	70	2 376	22.98	11.81	1.95	4.33		0.405	4.64	8.0	0.0884	55	
69	$\frac{3}{4}$	$1\frac{3}{4}$	$2\frac{7}{8}$	70	1 681	29.76	8.11	3.57	7.90	mixed	0.536	6.14	11.1	0.0943	69	
70	$\frac{3}{4}$	$1\frac{3}{4}$	$2\frac{7}{8}$	80	1 330	30.64	6.60	4.65	10.20	0.40	0.562	6.43	13.8	0.1061	68	
71	$\frac{3}{4}$	$1\frac{3}{4}$	$2\frac{7}{8}$	90	1 100	32.83	5.51	5 96	13 20	and	0.613	7.02	16.2	0.1179	71	
72	$\frac{3}{4}$	$1\frac{3}{4}$	$2\frac{7}{8}$	90	1 305	25.80	6.54	3.95	8.78	0.35	0.471	5.43	13.9	0.1179	55	
73	$\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{3}{4}$	70	1 956	21.68	9.68	2.24	4.98		0.386	4.42	9.6	0.1002	48	
73*a*	$\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{3}{4}$	70	1 154	22.72	5.71	3.98	8.84	0.26	0.423	4.84	15.7	0.0707	63	
74	$\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{3}{4}$	70	1 821	21.38	9 00	2.37	5.27	0.42	0.383	4.39	10.3	0.1002	48	
75	$\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{3}{4}$	70	1 950	21.58	9.62	2.24	4.96	0.42	0.385	4.41	9.7	0.1061	47	
76	$\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{3}{4}$	70	2 300	19.93	11.33	1.76	3.91	0.42	0.352	4.03	8.3	0.1002	44	
77	$\frac{3}{4}$	$1\frac{9}{16}$	$2\frac{3}{4}$	70	1 814	20.24	9.00	2.25	5.00	0.42	0.362	4.14	10.3	0.1002	45	
78	$\frac{3}{4}$	$1\frac{9}{16}$	$1\frac{3}{4}$	100	1 106	26.33	5.47	4.81	10.70	0.42	0.492	5.63	16.3	0.1238	55	
79	$\frac{13}{16}$	$1\frac{1}{4}$	$1\frac{7}{16}$	80	1 551	25.58	7.68	3.33	7.40	0.42	0.463	5.30	12.0	0.1002	58	
80	$\frac{13}{16}$	$1\frac{3}{4}$	$1\frac{7}{16}$	90	1 292	27.30	6.88	4.27	9.49	0.42	0.503	5.76	14.1	0.1061	61	

Ejecting sand through 402.4 ft. 3-in pipe. Lift, 1 267 ft. Delivery pipe level.

81	$\frac{3}{4}$	$1\frac{7}{16}$	$1\frac{3}{4}$	70	1 099	21.38	5.43	3.94	8.75	0.42	0.400	8.15	16.4	0.1325	89	
82	$\frac{3}{4}$	$1\frac{7}{16}$	$1\frac{1}{2}$	80	1 033	22.63	5.11	4.43	9.85	0.42	0.426	8.68	17.3	0.144	92	
83	$\frac{3}{4}$	$1\frac{7}{16}$	$2\frac{1}{4}$	80	1 000	22.80	4.96	4.60	10.22	0.42	0.431	8.78	17.7	0.144	93	
84	$\frac{3}{4}$	$1\frac{1}{4}$	$2\frac{1}{4}$	80	1 016	22.71	5.04	4.51	10.00	0.42	0.429	8.74	17.5	0.144	93	
85	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{3}{4}$	80	950	22.34	4.70	4.75	10.55	0.42	0.425	8.66	18.7	0.144	92	
86	$\frac{3}{4}$	$1\frac{1}{4}$	$1\frac{3}{4}$	70	1 036	21.09	5.13	4.11	9.13	0.42	0.397	8.09	17.2	0.1325	89	
87	$\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{1}{4}$	80	1 060	22.63	5.26	4.31	9.58	0.42	0.425	8.66	16.9	0.144	92	
88	$\frac{3}{4}$	$1\frac{3}{32}$	$1\frac{1}{4}$	80	963	23.09	4.78	4.83	10.73	0.42	0.438	8.92	18.3	0.144	94	
89	$\frac{3}{4}$	$1\frac{1}{16}$	$1\frac{1}{2}$	70	1 010	21.40	5.01	4.27	9.98	0.42	0.404	8.23	17.6	.....	..	
90	$\frac{3}{4}$	$1\frac{1}{16}$	$2\frac{1}{4}$	70	992	21.03	4.93	4.28	9.51	0.42	0.398	8.11	17.5	.....	..	
90*a*	$\frac{3}{4}$	$1\frac{1}{16}$	$2\frac{1}{4}$	70	968	23.12	4.77	4.84	10.74	0.26	0.439	8.94	18.3	.....	..	
91	$\frac{13}{16}$	$1\frac{1}{16}$	$2\frac{1}{4}$	70	1 130	24 18	5.59	4.32	9.60	0.42	0.451	9.19	15.9	.....	..	
92	$\frac{13}{16}$	$1\frac{1}{16}$	$2\frac{1}{4}$	70	1 179	24.28	5.84	4.16	9.25	0.42	0.451	9.19	15.4	.....	..	
93	$\frac{13}{16}$	$1\frac{1}{4}$	$2\frac{1}{4}$	70	1 206	24.36	5.97	4.08	9.07	0.42	0.451	9.19	15.1	.....	..	

gruel at a velocity of about 2 ft. per sec. With the 3-in. pipe, and velocities of about 8.5 ft. per sec., the sand was well suspended and the flow was quite satisfactory.

From the data secured on the pressures at the head of the delivery pipe, computations have been made for the value of c in the

Chezy formula, $v = c\sqrt{r\,s}$. For Experiments 34 to 48, the value of c varied from 63 to 76, the average being about 66. In Experiments 68 to 80, the value of c varied from 44 to 71, which is probably accounted for by the rolling of the sand and ponding at low velocities. In Experiments 81 to 88, where a good mixture was secured, the values of c are quite consistent, varying directly with the velocity; they indicate, for the first time in the tests, that variations were occurring in a regular manner. In the wide gaps of the 52% mixture with the $\frac{7}{16}$-in. nozzle, using water at 60 lb. pressure, giving 2-ft. velocities in a 4-in. pipe with good flow for short lift and delivery, as compared with the 18% mixtures at 9-ft. velocities, when the lift was a little more than doubled and the pipe was greatly increased in length, there is a good opportunity for experiments. Also, it would be interesting to get data when the lift is made at some distance from the ejector, as it appears that the impact given to particles of sand when the lift is made at the head of the delivery line materially affects the efficiency of the ejector. Mr. Vogleson.

In the winter of 1904-05, when stationed at the Belmont and Roxborough Filter Plants at Philadelphia, opportunity was obtained for detailed observation of the ejector system in operation. An important feature is the rapidity with which the throats wear out, and the necessity for a wearing part that is cheap and readily replaced. In the search for a satisfactory throat, chilled iron, steel, lead, copper, wood and rubber, were each given a trial. A removable piece having threads on it is unsatisfactory, because the sand gets into the threads. A slip-joint which can be cleaned with a dash of water appears to be quite satisfactory. Once, when the stock of regular throats had been exhausted, and the efficiency of the ejectors had dropped materially, the foreman of the filter attendants made a bushing from a piece of gas-pipe and a coupling. It fitted snugly in the worn throat, and was so successful that a supply of "pipe throats" was provided at practically no cost, and when needed they were quickly placed. It was also noted that the sections of hose used for sand deteriorated very rapidly, and the wisdom of keeping down the pressure, aside from considerations of pumping, became more apparent. An arrangement of permanent pipes, such that the shortest possible length of hose will be used, is also quite desirable. The number of gallons of water per cubic yard of sand ejected in actual operation is greater than was shown to be necessary by the experiments. This is in part due to the fact that in operation it is desirable to have the fewest possible number of stops in the flow, and there are times when the ejector is using water, but doing no useful work.

The sand dealt with at Philadelphia, when dirty, was very sticky, and its arching, well toward the top of the hopper, was very nicely

Mr. Vogleson.

TABLE 17.—CHANGE IN EFFECTIVE SIZE OF SAND CAUSED BY REPEATED EJECTING.

Number of sample.	Number of times ejected.	Weight analyzed, in grams.	Effective size, in millimeters.	U. C.	60% finer than mm.	PERCENTAGES PASSING SIEVES HAVING FOLLOWING SEPARATIONS, IN MILLIMETERS.									
						0.165	0.205	0.245	0.300	0.380	0.450	0.580	0.930	2.15	5.60
493	0	426.9	0.26	1.50	0.39	1.0	.....	6.87	26.0	54.3	78.8	93.9	99.6	....	100
491	0	455.2	0.26	1.50	0.39	0.7	2.7	6.50	25.8	54.1	78.9	93.4	99.2	....	100
492	1	475.7	0.23	1.48	0.34	3.1	.....	12.23	39.1	76.4	93.9	99.2	100.	....	....
494	1	416.0	0.255	1.39	0.355	0.8	.....	7.2	30.8	70.6	93 8	99.3	100.	....	....
495	1	415.7	0.26	1.50	0.39	1.2	.....	7.3	26.8	55.6	79.8	93.6	99.5	99.9	100
496	2	459.8	0.265	1.49	0.395	0.4	.....	5.3	24.4	53.7	80.1	94.1	99.5	99.9	100
497	3	398.5	0.26	1.50	0.39	0.7	.....	6.5	28.0	57.0	81.1	94.0	99.4	99.9	100
498	4	384.5	0.27	1.48	0.40	0.4	.....	4.9	23.4	51.7	77.8	92.8	99.3	99.8	100
499	5	414.9	0.27	1.48	0.40	0.6	.....	5.3	23.0	51.2	77.2	93.0	99.4	99.9	100
500	6	416.1	0.27	1.48	0.40	0.2	.....	4.0	22.1	51.8	77.8	93.2	99.5	99.9	100
587	7	425.9	0.27	1.48	0.40	0.3	.....	4.4	21.2	51.2	76.4	93.0	99.4	99.9	100
588	15	396.7	0.27	1.48	0.40	0.1	.....	4.3	22.4	51.2	75.9	91.7	98.5	99.6	100
589	22	436.5	0.275	1.49	0.41	0.2	.....	3.6	19.3	47.8	72.0	91.0	98.6	99.8	100

Samples 492 and 494 were taken from measuring box when sand and water were discharged directly into it. Samples 495 to 589 were taken when the waste water was flumed across box. Sand shoveled from flume to box.

TABLE 18.—COMPARISON OF COST OF CLEANING FILTERS AT PHILADELPHIA, ALBANY AND WASHINGTON.

Items.	LOWER ROXBORO'.		UPPER ROXBORO'.		BELMONT.		ALBANY.	Washington.
	1904	1905	1904	1905	1904	1905	‡1899–1901	
Average cost of scraping, per cubic yard	$0.21	$0.19	$0.26	$0.18	$0.27	$0.21	$0.13	$0.05
Average cost of removing, per cubic yard	0.27	0.25	0.30	0.22	0.28	0.23	0.25	0.16
Average cost of washing, per cubic yard	0.20	0.15	0.15	0.09	0.12	0.09	0.30	0.05
Average cost of restoring, per cubic yard	*0.49	†0.30	*0.52	†0.30	*0.50	†0.27	0.25	0.13
Total cost of cleaning, per cubic yard	$1.17	$0.89	$1.23	$0.79	$1.17	$0.80	$0.93	$0.39
Average cost per million gallons to scrape, transport and wash	0.668	0.521	0.93	0.39	1.086	0.825	1.01	.....§
Cost of transporting, washing and restoring, per million gallons filtered	1.13	0.79	1.85	0.63	1.89	1.25	1.66	0.60

* Restored by barrows—day labor.
† Restored by barrows—contract.
‡ Mr. Gregory, *Transctions*, Am. Soc. C. E., Vol. XLVI, p. 351.
§ Average of the figures given by the authors for cost of cleaning per million gallons filtered.

At Philadelphia, laborers are paid $1.75 for 8 hours' work.
At Albany and Washington, laborers are paid $1.50 for 8 hours' work.
At Philadelphia, in 1904, wash-water was charged at $25 per million gallons.
At Philadelphia, in 1905, wash-water was charged at $15 per million gallons.

overcome by the jet sprinkler shown on Fig. 19, by reason of the jets striking into the arch, breaking it down, and at the same time preserving the water level in the hopper at a low elevation, which was found to be desirable for rapid work. Mr. Vogleson.

The unit costs for sand handling at Washington show marked reduction in that feature of operation. From reports of the Philadelphia Bureau of Filtration for 1904 and 1905, and from the detailed analysis of costs of operation at the Albany Filters,* given by John H. Gregory, Assoc. M. Am. Soc. C. E., the data in Table 18 have been collected.

In the figures for Philadelphia, the entire cost for labor, from the time of entering the filter to clean it until it was in shape to begin back-filling, is distributed under the headings of Scraping, Removing and Washing.

At Philadelphia the cost of scraping includes the time required for screeding. Removing included all time required for connecting up and later storing the ejectors and all tools. Washing, in 1904, for the greater part of the year, was carried on without sand bins, and extra labor was required to keep the sand out of the sewers, which to some extent accounts for the reduction in cost of washing in 1905. It will be noted that the costs for sand handling at Philadelphia are being reduced from $1.20 per cu. yd., as stated by the authors, which applies to the average of $1.19 for 1904. The average cost for 1905 was 83 cents per cu. yd.

CHARLES MOORE, ESQ.—The speaker was concerned in the investigation of the question as to the proper filtration system for Washington; in the selection of the commission (composed of Messrs. Hering, Fuller and Hazen) which reported in favor of the slow sand system that was afterward installed by direction of Congress; and in the choice of Mr. Hazen as Consulting Engineer, and, naturally, has been somewhat disturbed by the rumors reflecting on the efficiency of the completed plant. Therefore, it was with much satisfaction that he had listened to the able discussion, for all his doubts had been removed. Without question, the bacterial efficiency of the plant is established; and the prevalence of typhoid fever in Washington must be accounted for in one of the numerous other ways suggested by the discussors. Indeed, the fact was recognized from the beginning that the unregulated milk supply, the primitive sanitary arrangements (or lack of them) in certain quarters of the city, and the continued use of wells, were all sources of contagion. Mr. Moore.

It is to be hoped, however, that the Washington filtration plant may be completed speedily according to the original report. Until that final step is taken, by the use of a coagulant added at a point

* *Transactions*, Am. Soc. C. E., Vol. XLVI, p. 351.

Mr. Moore. near the Great Falls intake, during periods of greatest turbidity (approximately one-twelfth of the time), it is not fair to the engineers who planned the system to hold them responsible for the occasional overloading of the filter beds. That is a result they foresaw clearly and provided against. The responsibility, therefore, for any failure to reach the very best results must remain with those who have hesitated to complete the plant according to the original plans. This hesitation seems to be unwarranted, short-sighted, and wasteful.

Miss Blatch. NORA STANTON BLATCH, JUN. AM. SOC. C. E. (by letter).—In connection with the description of the removal of the sand for cleaning, the authors give some information on the laws governing the flow of sand and water in pipes under pressure. As little is known on this subject, an outline of the results of an experimental investigation along this line (made by the writer at the Hydraulic Laboratory of Cornell University), and a comparison with the authors' conclusions may be of interest.

Three series of experiments were made, as follows:

Series 1.—With a 1-in. brass pipe (actual diameter 1.0546 in.), and a coarse grade of sand, called Grade I. This sand lay between sieves of 20 and 40 mesh, and its specific gravity was 2.64.

Series 2.—With the same pipe and a fine grade of sand, called Grade II. This grade lay between sieves of 60 and 100 mesh. Its specific gravity was also 2.64.

Series 3.—With a 1-in. galvanized-iron pipe (actual diameter 1.0420 in.), and Grade I sand.

Tests were also made to determine the loss of head in the brass and galvanized-iron pipes with water alone flowing in them.

A horizontal pipe, about 27 ft. long, was set up on standards. By means of a valve at one end, water from a tank could be admitted at a constant pressure of 20 lb. A tee was inserted in this pipe about 2½ ft. from the valve. The sand-feeding apparatus was a water-tight cylinder, connected to this tee by a rubber hose. The cylinder was refilled with sand after every two or three runs. Water-jets in the lower part of the cylinder prevented the sand from clogging. By varying the discharge of these jets, and by regulating the entrance valve on the main pipe, the velocity and the percentage of sand could be controlled. From the tee to the up-stream piezometer a length of 6½ ft. was allowed in order to insure steady flow. The experimental length between the piezometers was about 12 ft., on both the brass and the galvanized-iron pipes; and the discharge was 4½ ft. from the down-stream piezometer. The loss of pressure head in the experimental length was read from a differential mercury gauge; and a stop-cock was attached at the discharge so that the zeros of the gauges could be tested frequently

Miss Blatch.

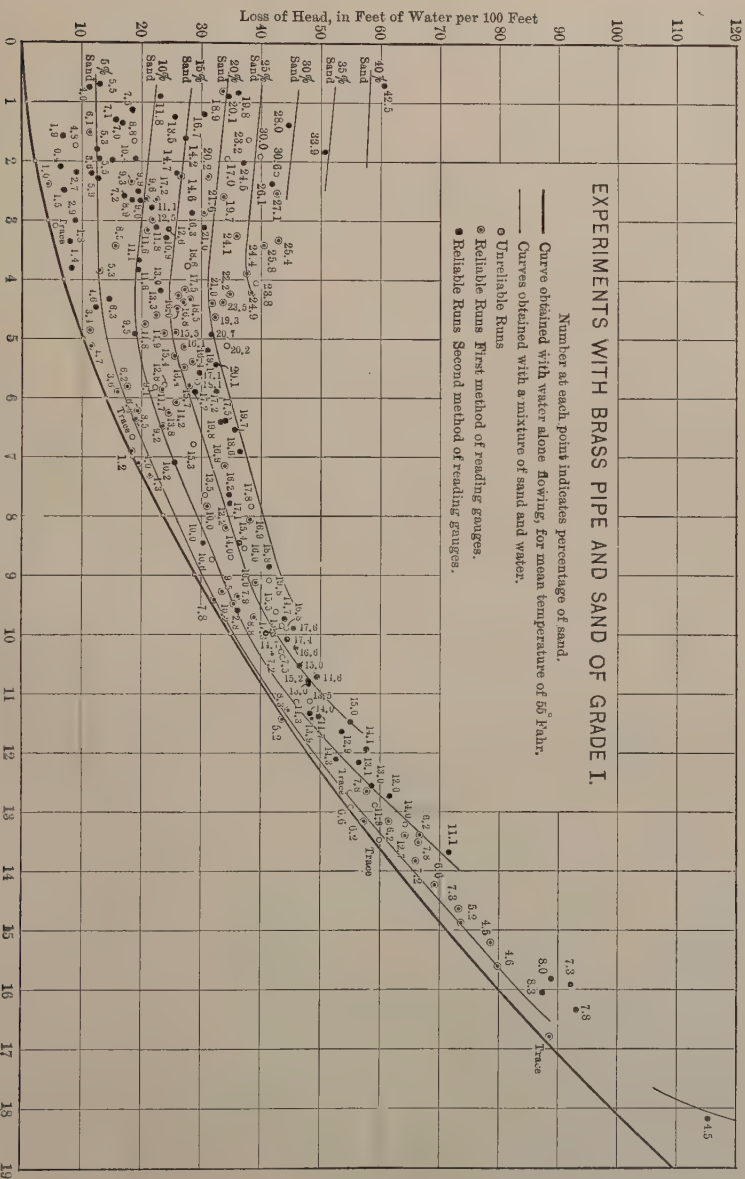
EXPERIMENTS WITH BRASS PIPE AND SAND OF GRADE I.
Number at each point indicates percentage of sand.
Curve obtained with water alone flowing, for mean temperature of 50° Fahr.
Curves obtained with a mixture of sand and water.
Unreliable Runs
Reliable Runs First method of reading gauges.
Reliable Runs Second method of reading gauges.
Loss of Head, in Feet of Water per 100 Feet
Mean Velocity, of Mixture of Sand and Water, in Feet per Second

FIG. 22.

Miss Blatch. under static head. A rubber hose was used to deflect the flow from the waste tank to the measuring tank. The latter stood on scales, so that the total weight of sand and water discharged during a given time could be noted, and thus the velocity determined. Then, by measuring the volume of sand and knowing the percentage of voids, the percentage of sand could be computed.

Throughout the writer's experiments, the percentage of sand was taken to equal the ratio of the volume of sand particles (not including the voids) to the volume of the mixture. The percentages given in Figs. 22, 23, and 24 are computed according to this definition. Both the Mississippi River Commission and the authors used the definition: percentage of sand equals the ratio of the volume of sand including voids to the volume of the mixture. The percentages of Fig. 25 and the quantities plotted in Fig. 26 are computed according to the latter definition.

The results of these experiments on small pipes were compared with the data obtained from tests of dredges made by the Mississippi River Commission.* In these tests, mixtures of sand and water were pumped through riveted-steel discharge pipes from 30 to 34 in. in diameter. The discharge pipes varied in length from 500 to 1 100 ft. The loss of head was measured by an open mercury gauge near the pump. The pipe discharged into a measuring barge for a given time, and then the volumes of sand and water together, and of sand alone were measured. In this way the velocity and percentage of sand were determined. The sand pumped was ordinary river sand having an average specific gravity of 2.54. It was hoped that, after determining the laws for 1-in. and 30-in. pipes, the laws for intermediate sizes could be approximated.

Figs. 22, 23, and 24 show plottings of the writer's experiments. Fig. 22 is a plotting of Series 1, and shows the general form of the curves obtained. Fig. 23 is a plotting of Series 2, with the resulting curves. The dotted lines are the curves of Series 1 superimposed for the purpose of comparing the effects of the two grades of sand. Fig. 24 is a plotting of Series 3. The curves of Fig. 22 are here again superimposed to show the relative effect of rough and smooth pipes on the loss of head, the grade of sand remaining the same.

From these three diagrams the writer draws the following conclusions for the conditions under which the experiments were performed:

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* For the description and data of these tests, see Report, Chief of Engineers, U. S. Army, for 1895 and 1898.

Annual Report of the Miss. River Comm., 1903, Appendix I-F; and 1904, Appendix I-D.

"Dredges and Dredging on the Mississippi River," by J. A. Ockerson, *Transactions*, Am. Soc. C. E., Vol. XL, p. 215.

"Hydraulic Dredging on the Mississippi River," by F. B. Maltby, *Transactions*, Am. Soc. C. E., Vol. LIV, Part C, p. 391.

1.—Two distinct conditions of flow exist; one obtains for low velocities, up to about 3½ to 4 ft. per sec., and the other for high velocities beginning between 4 and 10 ft. per sec. Between these two conditions of flow is a transition period, which is short if the sand is fine and uniform, and long if there is a great mixture of grades. Miss Blatch.

2.—The total loss of head due to any given mixture of sand and water is almost constant for low-velocity flow. For the transition period it increases with a lower power of the velocity than the loss of head due to water alone (that is, less than the 1.74 power of the velocity). For high-velocity flow, the indications seem to be that the loss of head increases with a higher power of the velocity than 1.74.

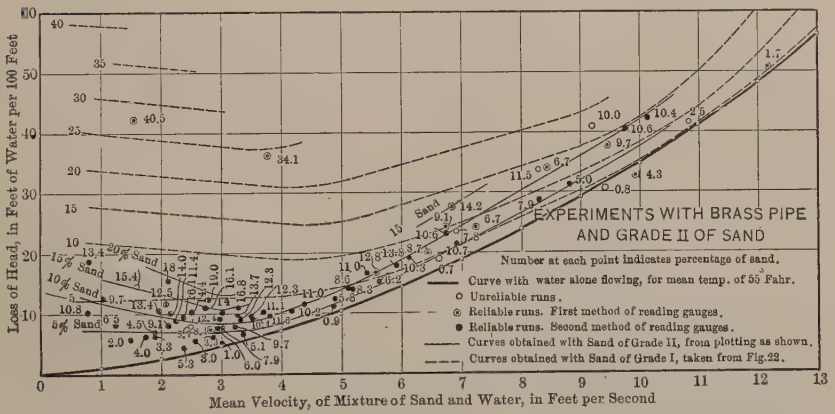
Loss of Head, in Feet of Water per 100 Feet
Mean Velocity, of Mixture of Sand and Water, in Feet per Second
EXPERIMENTS WITH BRASS PIPE AND GRADE II OF SAND
Number at each point indicates percentage of sand.
Curve with water alone flowing, for mean temp. of 55° Fahr.
Unreliable runs.
Reliable runs. First method of reading gauges.
Reliable runs. Second method of reading gauges.
Curves obtained with Sand of Grade II, from plotting as shown.
Curves obtained with Sand of Grade I, taken from Fig. 22.
5% Sand
10% Sand
15% Sand
20% Sand
Sand

FIG. 23.

3.—The excess loss of head, over that when water alone is flowing, is greatest for low velocities and least just where high-velocity flow sets in. For the first condition of flow, with more than 5% of sand, the loss of head is constant for equal added percentages. The loss of head for each 1% of sand is approximately as follows:

12 ft. per 1 000 ft.—Brass pipe.—Grade I sand.—Series 1.
4 ft. per 1 000 ft.—Brass pipe.—Grade II sand.—Series 2.
14 ft. per 1 000 ft.—Galvanized-iron pipe.—Grade I sand.—Series 3.

For the first 5% of sand the excess loss of head is greater for the lowest velocity investigated, but decreases to the above values at about 3 ft. per sec. For velocities greater than 3½ ft. per sec. the loss of head due to low percentages becomes very small, while the increment of head due to equal added percentages of sand increases.

Miss Blatch.

4.—In Fig. 23, it is seen that the loss of head due to fine sand for any given percentage and velocity below about 7 ft. per sec. is less than that due to a coarse sand. For higher velocities, the loss of head seems to be greater for the fine sand, up to the highest velocity investigated.

5.—From Fig. 24 it may be concluded that the total loss of head is greater at all velocities for the galvanized-iron than for the brass pipe. The excess loss of head due to the sand is greater in the rough pipe at low velocities; rather less at medium velocities, and about the same at high velocities, as the loss of head due to sand alone in the smooth pipe.

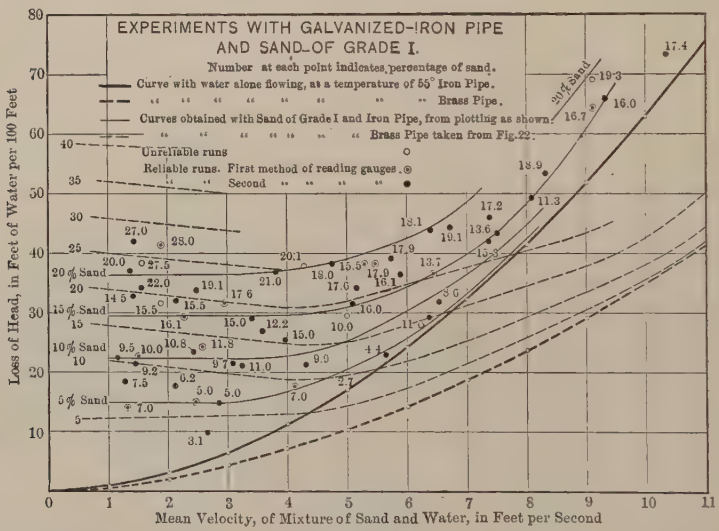
EXPERIMENTS WITH GALVANIZED-IRON PIPE AND SAND OF GRADE I.
Number at each point indicates percentage of sand.
Curve with water alone flowing, at a temperature of 55° Iron Pipe.
" " " " " " " " " Brass Pipe.
Curves obtained with Sand of Grade I and Iron Pipe, from plotting as shown
" " " " " " " " Brass Pipe taken from Fig. 22.
Unreliable runs
Reliable runs. First method of reading gauges.
" " Second " " " "
20% Sand
15% Sand
10% Sand
5% Sand
Loss of Head, in Feet of Water per 100 Feet
Mean Velocity, of Mixture of Sand and Water, in Feet per Second

FIG. 24.

Considering one size of pipe, the velocity at which any given volume of sand per hour can be transported through a given length of pipe, with the least expenditure of power per unit volume of sand carried, will be called the economical velocity. In other words, the economical velocity is that at which the loss of head due to any given mixture of sand and water is a minimum.

6.—Since the loss of head, due to any given mixture of sand and water, for both grades of sand and both kinds of pipe,* is least at from 3 to 4 ft. velocity, about 3½ ft. per sec. is the most economical velocity for a 1-in. pipe.

* In Fig. 24 the curves obtained with the galvanized iron pipe and the coarse grade of sand are horizontal below 3½ ft. per sec., so that the economical velocity might be considered as located anywhere below 3½ ft. per sec., as well as at that point.

7.—At any given velocity, the higher the percentage of sand the more economical is the process. This is true for both grades of sand and both kinds of pipe. Miss Blatch.

In order to observe directly the nature of the flow of the sand and water, short glass sections* were inserted in both kinds of pipe.

It was found that, for velocities of less than about 3½ ft. per sec., the greater part of the sand was dragged along the bottom of the pipe at a very low velocity, practically clear water flowing in the upper part of the cross-section. At a velocity of about 3½ ft. per sec. large quantities of sand began to be carried in suspension. (It should here be noted that under Conclusion 4 it was stated that the economical velocity occurred at this point.) This change occurred suddenly for the fine grade of sand, while for the coarse grade, in which the particles differed more in size, the change was gradual. At the higher velocities, all the sand was carried in suspension throughout the cross-section.

It is interesting to note that these three stages correspond to the low-velocity flow, the transition period, and the high-velocity flow, as mentioned in Conclusion 1. Thus the cause of the very high loss of head at low velocities is due to the dragging of the sand along the bottom of the pipe and to the small cross-section available for the flow of the water. The gradual decrease in the loss of head during the transition period is due to the gradual suspension of the sand. At high velocities the loss of head due to the sand is small on account of all the sand being suspended.

Fig. 25 is a plotting of the dredge experiments of the Mississippi River Commission, already mentioned. Although the curves corresponding to those of Figs. 22, 23 and 24, can be only roughly approximated, a few facts can be gained from them. Considering the roughness of the discharge pipes and the quality of the sand dredged, the curves of Fig. 25 should be compared with the curves of Fig. 24, based on experiments with the galvanized-iron pipe and the coarse grade of sand.

The great irregularity of the tests with the dredge *Beta* is probably due to the fact that two discharge pipes were used simultaneously and the mean of the gauge readings taken, although, in most cases, the velocities and percentages of sand in the two pipes differed greatly. The curves of the dredge *Gamma* would seem to indicate that the economical velocity was about 9 ft. per sec., while, on the dredge *Alpha,* the economical velocity cannot be estimated.

When, in the dredge experiments, the velocity was allowed to drop 2 or 3 ft. below 9 ft. per sec., the cross-section became par-

* The glass sections were slightly less than 1 in. in diameter, and were connected to the main pipe by rubber hose and wire. They were inserted up and down stream from the experimental length, so as not to interfere with the gauge readings.

Miss Blatch. tially blocked, owing to large quantities of sand settling in the lower half of the pipe. Since, on 1-in. pipes, the economical velocity occurred at the point where large quantities of sand were first carried in suspension, it would seem that about 9 ft. per sec. was the economical velocity for the dredge experiments. This statement agrees with the indications of the curves for the dredge *Gamma*.

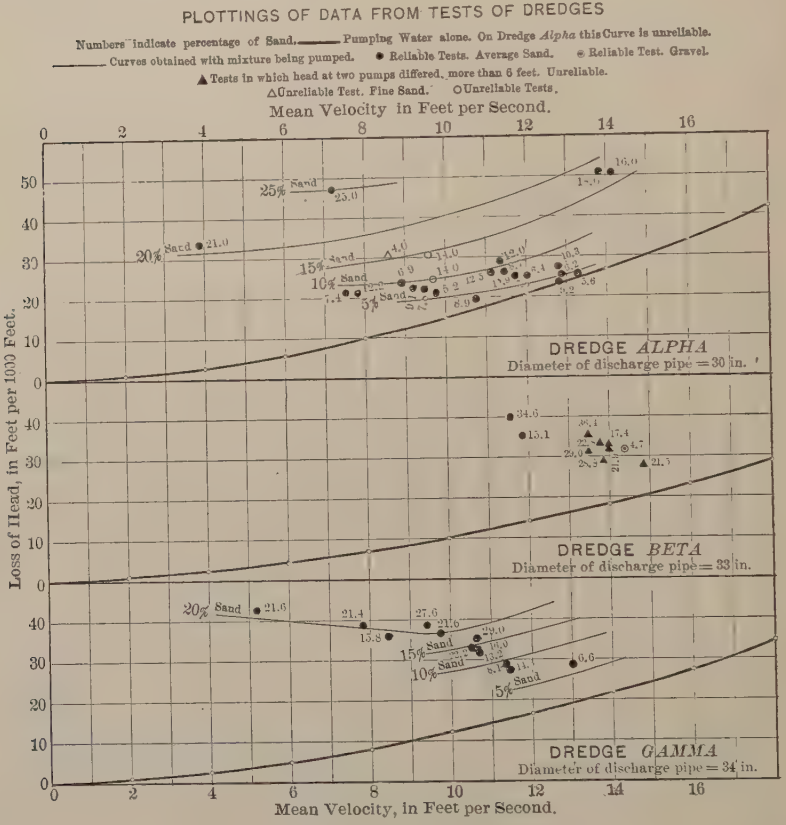
PLOTTINGS OF DATA FROM TESTS OF DREDGES
Numbers indicate percentage of Sand. ——— Pumping Water alone. On Dredge Alpha this Curve is unreliable.
——— Curves obtained with mixture being pumped. ● Reliable Tests. Average Sand. ⊙ Reliable Test. Gravel.
▲ Tests in which head at two pumps differed more than 6 feet. Unreliable.
△ Unreliable Test. Fine Sand. ○ Unreliable Tests.
Mean Velocity in Feet per Second.
Loss of Head, in Feet per 1000 Feet.
DREDGE ALPHA
Diameter of discharge pipe = 30 in.
DREDGE BETA
Diameter of discharge pipe = 33 in.
DREDGE GAMMA
Diameter of discharge pipe = 34 in.
25% Sand
20% Sand
15% Sand
10% Sand
5% Sand
Mean Velocity, in Feet per Second.

FIG. 25.

The economical velocity, therefore, varies from about 3½ ft. per sec. on 1-in. pipes to about 9 ft. per sec. on 32-in. pipes,* but, whether or not this velocity varies directly with the diameter of the pipe cannot be determined without knowing the results with some intermediate sizes.

* The mean diameter of the discharge pipes of the dredges, *Alpha*, *Beta* and *Gamma* is 32 in.

Approximate rules, for finding the loss of head for any given percentage of sand at the economical velocity, may now be given, the quantities being estimated from the curves. Miss Blatch.

For pipes 1 in. in diameter:

$$\left.\begin{matrix}\text{Total loss of head, in} \\ \text{feet per 1 000 ft.}\end{matrix}\right\} = \left\{\begin{matrix}\text{Loss of head due to water alone at } 3\tfrac{1}{2} \\ \text{ft. per sec. + 9.6* ft. per 1 000 ft.} \\ \text{for each 1\% of sand.}\end{matrix}\right.$$

For pipes 32 in. in diameter:

$$\left.\begin{matrix}\text{Total loss of head, in} \\ \text{feet per 1 000 ft.}\end{matrix}\right\} = \left\{\begin{matrix}\text{Loss of head due to water alone at 9 ft.} \\ \text{per sec. + 1.2 ft. per 1 000 ft. for} \\ \text{each 1\% of sand.}\end{matrix}\right.$$

Having stated these conclusions concerning 1-in. and 32-in. pipes, a comparison with the authors' results, using 3-in. and 4-in. pipes, will be of interest.

In Fig. 13 the curves follow one general law for all velocities and all percentages of sand. As no plotted points are shown, one cannot tell how closely they follow the observed facts. The curves correspond fairly well with those of the writer for high velocities, as both run approximately parallel to the curve obtained when water alone is flowing. For low velocities, however, the curves differ radically in every way.

The effective cross-sections for the various sizes of pipe are compared in Table 19. The results obtained with sand of Grade I and the galvanized-iron pipe are given for the 1-in. pipe.

TABLE 19.

Diameter, in inches.	Velocity, in feet per second, at which flow is often blocked.	Minimum Velocity for steady flow; or economical velocity, in feet per second.	Velocity, in feet per second, at which all sand is suspended.
1	1¼	3½	8 to 9
3	2½	4	.....
32	6 or 7	9	14

The authors give this approximate rule† for finding the loss of head in a 3 or 4-in. pipe with any given velocity, and percentage of sand flowing.

$$\left.\begin{matrix}\text{Total loss of head, in} \\ \text{feet per 1 000 ft.}\end{matrix}\right\} = \left\{\begin{matrix}\text{Loss of head due to water alone at the} \\ \text{given velocity + 15 ft. per 1 000 ft.} \\ \text{(constant) + 3 ft. per 1 000 ft. for} \\ \text{each 1\% of sand.}\end{matrix}\right.$$

* The writer uses 9.6 ft. instead of 14 ft. as given under Conclusion 3, owing to the different definitions of the percentage of sand, as previously stated. The average percentage of voids in Grade I = 46.2.

† In some additional information on the experiments performed at Washington.

Miss Blatch. The constant would appear to correspond to the additional loss of head for the first 5% of sand referred to under Conclusion 3 of the writer's experiments, but this latter disappears at 3½ ft. per sec., whereas the authors' constant is to be used for all velocities.

Concerning the increment of loss of head alone, the comparison in Table 20 can be made.

TABLE 20.

Diameter, in inches.	Loss of head, in feet per 1 000 ft. due to 1% of sand.
1	9.6
3 and 4	3
32	1.2

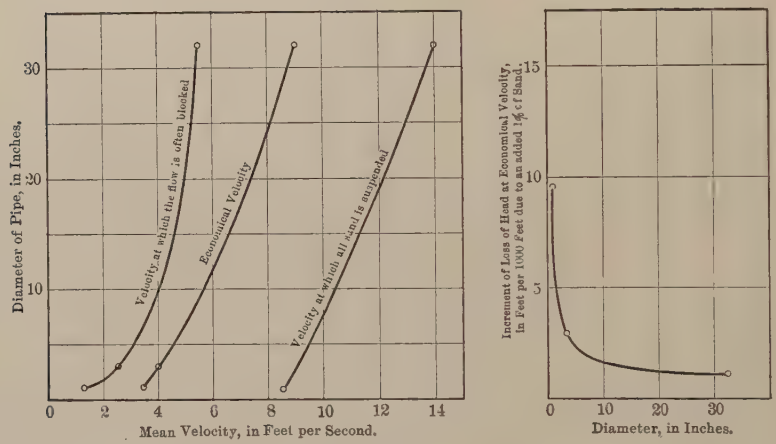

FIG. 26.

The data from Tables 19 and 20 are plotted in Fig. 26. Although the curves there shown are only roughly approximated for pipes ranging from 4 to 30 in. in diameter, they serve to show the general trend of results for all sizes of pipe up to 32 in. in diameter. The writer thinks that, by their means, approximations can be made as to the size of pipe and the most economical working velocity that should be used under given conditions.

Prof. Sedgwick. W. T. SEDGWICK,* ESQ.—The speaker is strongly of the opinion that the Washington filters, both in construction and in operation, are highly efficient; and that the typhoid in the city is of what he has elsewhere described as residual (*i. e.*, non-water-borne) origin.

*Professor of Biology, Massachusetts Institute of Technology.

H. F. LABELLE, M. AM. SOC. C. E. (by letter).—The St. Cloud Heights Reservoir, the most recent of the three distributing reservoirs of Paris, France, is a typical one, as regards covered reservoirs. It is the largest structure of this kind in the French capital, and the writer believes it is considered by French engineers as making a record in boldness of design.* Mr. Labelle.

The structure is entirely of brick masonry in cement mortar. The piers are not more than 16 by 16 in.; the span of the arches is about 15 ft., and the height of the piers to the spring of the arch is 17 ft., or about thirteen times their width.

The first bay adjoining the outside walls is filled by a cross-wall to the first row of piers; an arched opening, 5 ft. wide, is provided in this wall for economy's sake and to favor the circulation of water. This cross-wall receives the thrust of the ground arches in the next bay; it is not bonded with the outside wall, but stands about ½ in. from it. Circular arches are thrown over the cross-walls, and are groined at the second bay and run over the outside walls at the other end.

These arches (and the writer presumes all the arches in the structure) have concentric intrados and extrados, and are built of two rings of brick, laid flat; the brick is about 1½ in. thick, so that, allowing ½ in. for the joints, the arch is about 3½ in. in thickness, uniformly, from spring to crown. The rise is about one-fifth of the span. The arches have the usual quantity of earth covering and also glass-covered manholes. Spiral staircases are provided at different points.

The outside walls are about 18 in. thick at the top, and are plumb in the inside of the reservoir. Unfortunately, the writer has not been able to ascertain the thickness at the bottom of the wall; it is probable, however, that there, also, the design is very economical. At any rate, the outside walls which have to support the embankment, have not come in contact with the cross-walls above referred to at places inspected by the writer.

In the design of this reservoir, the system of groined arches is independent of the outside walls, which do not receive any thrust whatever from the inside when the reservoir is empty.

As the outside walls of a covered reservoir are an important part of the whole masonry, the writer believes that considerable saving could be made on the so-called gravity sections by substituting for them armored-concrete buttressed walls, in connection with the cross-wall design of the St. Cloud Reservoir.

In 1903 the writer had occasion to design a new system of water supply for Cienfuegos, Cuba, and a concrete covered reservoir formed part of the works. Comparative estimates having been made, between an all-concrete and an armored-concrete structure,

* The reservoir is said to have been designed by Belgrand.

Mr. Labelle. it was found that an economy of $50 000, or 20%, could be obtained by the adoption of the latter design. At that time the designing of concrete-steel structures was not as well understood as it is at the present time, and the writer now realizes that he was ultra-generous in the quantity of steel used, and that at least $10 000 more could have been saved on this item, without impairing the stability of the structure.

The 72-in. Venturi meter in concrete, described by the authors, is certainly a step forward in the evolution of this eminently useful device.

Dr. Magruder. G. LLOYD MAGRUDER, M. D.—The speaker has taken great interest in the Washington water supply, especially with reference to water-borne diseases. As Mr. Longley has stated, Dr. Kober, in a paper read in Berlin in 1890*, first called the attention of the profession to the Potomac water supply as being the probable cause of an increase in typhoid fever.

In February, 1894, the late Dr. Smart, of the Army, who was recognized as a leading sanitarian and authority upon water purification and analysis, read a paper before a society organized in Washington for the purpose of disseminating knowledge in regard to the sanitary condition of the city.

The city was in a deplorable condition, the sanitary regulations were few, limited, and in many cases primitive, and there were no regulations in regard to milk. Dr. Smart's paper dealt largely with the Potomac water supply, and attributed the high death rate from typhoid and typho-malarial fever to its condition. The public being invited to take part in the discussion, the speaker took issue with Dr. Smart, maintaining that there were other causes present, and that the city death rate given by Dr. Smart was probably greater than the city deserved. Subsequent investigations sustained Dr. Smart's contention as to the number of deaths. Two days later, at a meeting of the Medical Society of the District of Columbia, the speaker called attention to Dr. Smart's paper, and asked if it was the experience of the profession that such a high death rate from typhoid fever had prevailed, as he had had very few cases during the previous year, and had frequently received letters from people throughout the country asking whether they could spend their winters in Washington with freedom from typhoid or malaria. Dr. Billings also took issue with Dr. Smart's statements, and said that he was of the opinion that other causes than Potomac water produced the diseases. As a result of the discussion, Drs. William W. Johnston and C. M. Hammett, and the speaker, as chairman, were appointed a committee to investigate the subject.

The committee's investigation extended over 5 months, and its

* Published in the *Transactions* of the International Medical Congress.

report, submitted to the Medical Society on June 6th, 1894, showed that there were several factors tending to produce these diseases: (*a*) the Potomac water supply; (*b*) the pollution of the soil with leakage from privies, from defective sewers, and from backing up of sewage from tidal action; (*c*) the drinking of well or pump water; and (*d*) contaminated milk. Dr. Magruder.

Among the recommendations of this committee—unanimously adopted by the Society—were: The construction of works for the filtration of the Potomac water, the only proper method of purification; the abandonment, as rapidly as possible, of all wells within the city limits; the repairing of defective sewers; the extension of water supply and sewers; the making of house connections to the sewers; careful inspection of all dairies in the District from which the milk supply is drawn; and the enactment of a law by which no milk shall be sold in the District without a permit from the Health Office. The inspection should cover an examination of all possible sources of infection at the dairies, including the water supply.

In making these investigations, it was found that, at that time, there were more than 200 wells in the city, and, through the kindness of the Surgeon-General of the Marine Hospital, Dr. Kinyoun made bacteriological tests of 13 of these wells, 10 of which showed sewage contamination; and he recommended that they be closed at once. It was found that the city authorities were engaged in relaying many of the sewers which had been constructed by the Board of Public Works. In a number of cases, where the sewers were being relaid, it was noticed that the joints of the old sewers were defective, due to improper construction. Sometimes only half the water passed through the sewer, the other half soaking into the adjoining soil; and in some cases the stream was actually traced running into wells.

The District Commissioners were so much impressed with the value of this report that the attention of the Committees on the District of Columbia of the Senate and the House of Representatives was called to it. The Medical Society was invited by these Committees to present this report, and it was first presented to the Committee of the House of Representatives. This Committee was also much impressed with the importance of the text, maps and diagrams, which were very exhaustive, and had it published as a Congressional document. Promptly following this presentation, much sanitary legislation was secured, including a law governing the milk supply.

Dr. Woodward, the present Health Officer, was appointed after this investigation. He has endeavored persistently to improve the conditions, and to supervise the milk supply, as well as he could with the limited means at his disposal.

Dr. Magruder.

In the Health Officer's report for 1895, a special report was made by Dr. Kober upon the typhoid fever occurring that year. All the conclusions of the committee, of which the speaker was a member, were indorsed. Further tests of the water supply were made. Dr. Walter R. Reed made bacteriological tests of a number of the wells of the District of Columbia, and found the same condition of sewage contamination as had been noted the year before. At that time the water supply of 6 dairy farms was examined; and 4 of them showed sewage contamination.

During the summer of 1906 there was an increase in the number of cases of typhoid fever, as compared with the same season of 1905. The public prints were teeming with articles condemning the water supply and questioning the efficiency of the filtration plant. Letters were received from different parts of the country asking what it meant, and why typhoid fever had increased to such a degree. Again realizing, as in 1894, the damage that these reports were doing to the city and the process of sand filtration, the speaker took occasion to consult Captain Spencer Cosby, United States Corps of Engineers, in charge of the Washington Aqueduct. Tables 21 and 22, kindly furnished by him, show the great efficiency of the plant. The report of the Lawrence beds is furnished for the sake of comparison.

TABLE 21.—PERCENTAGE OCCURRENCE OF *B. Coli* IN THE EFFLUENT OF THE LAWRENCE CITY FILTERS, LAWRENCE, MASS.

YEAR.	1899.	1900.		1902.		1903.		1904.	
Sample tested.	1 c.c.	1 c.c.	100 c.c.	1 c.c.	100 c.c.	1 c.c.	100 c.c.	1 c.c.	100 c.c.
January	54	41	78	4	10	0	0	4	46
February	62	11	78	4	17	4	13	13	38
March	8	8	..	6	6	8	8	0	45
April	17	0	0	0	0	8	8	0	25
May	0	40	56	0	0	0	18	0	25
June	0	17	50	0	0	0	60	0	60
July	14	0	25	0	0	25	75	25	25
August	0	20	20	14	10	20	20	40	..
September	0	14	0	0	7	0	9	0	50
October	0	0	0	0	0	0	4	0	20
November	24	12	10	0	0	9	13	11	37
December	20	38	58	8	16	4	4	4	28
Average percentage positive	24	18	38	4	8	4	13	8	33
Typhoid death rate per 100 000	20	11		11		23		11	

TABLE 22.—PERCENTAGE OCCURRENCE OF *B. Coli* IN WASHINGTON WATER SUPPLY. Dr. Magruder.

1906.	FILTER EFFLUENT.				TAP WATER.		
	$\frac{1}{10}$ c.c.	1 c.c.	10 c.c.	100 c.c.	$\frac{1}{10}$ c.c.	1 c.c.	10 c.c.
January	0	0	7	..	..	..	..
February	0	0	0	..	..	..	..
March	0	0	0	..	0	0	0
April	..	0	4	12	0	0	0
May	..	0	0	0	0	0	0
June	..	0	0	4	0	0	3
July	..	0	5	19	..	..	..
August	..	4	4	44	..	5	10

On June 30th, 1906, the District Commissioners, being very solicitous about the prevailing conditions, addressed a letter to Surgeon-General Wyman, of the Bureau of Public Health and Marine Hospital Service, and invoked his aid to ascertain the causes of the continued prevalence of typhoid fever, notwithstanding the completion of the filtration plant. The speaker, in conversation with the Surgeon-General, laid great stress upon the milk supply as being an important factor in the causation of the trouble, and urged a most thorough investigation of this subject, maintaining that the Potomac water supply did not materially influence the disease. Drs. Rosenau and Lumsden, and Mr. Kastle, of the Hygienic Laboratory, were appointed a commission to pursue these investigations. About August 10th, they reported to Surgeon-General Wyman that, up to that time, they found the principal source of infection to be the water supply, from the fact that they found *Colon Bacilli* in some of the samples of Potomac water drawn from taps and in many of the shallow wells. From this report the Surgeon-General suggested to the District Commissioners that the Potomac water should be boiled before being used for drinking.

Dr. Wyman's letter created quite an additional disturbance in the newspapers, and caused more editorials throughout the country. People seized the opportunity to criticise the water supply system, and various manufacturers of ozone purifiers, etc., took advantage of the occasion to praise their methods.

The speaker discussed the matter with the Secretary of Agriculture, drawing his attention to the record of the efficiency of the filtration plant, as shown by the bacteriological observations carried on by Mr. Longley. The Secretary agreed that the condition of the milk supply was a very serious source of infection, and as a result of these discussions, authorized a consultation with the

Dr. Magruder.

chiefs of certain divisions of his department for the purpose of investigating the questions and especially the efficiency of the filtration plant and the condition of the milk supply.

A very careful record of the bacterial examinations of the water supply of Washington had been kept, both before and after the installation of the filtration plant, and it happened on several occasions that there were very high counts of bacteria in the water drawn from the taps in two of the laboratories there. These high counts were far in excess of any that had been obtained at any time in the laboratory at the filtration plant, and gave good ground to doubt the efficiency of the plant. As there was a question in his mind, and among his scientists, it was decided that it would be very desirable to have a conference between Mr. Hardy and Mr. Longley, who were in charge of the filtration works, and the scientists of the Agricultural Department.

This conference—authorized by the Secretary—was held about September 22d, 1906, and it was agreed to institute a series of check experiments. The scientists of the Department of Agriculture had made their counts from agar plates, while those at the filtration plant had been made from gelatine plates. They agreed to go, day by day, and secure samples from the same sources, use the same media, exchange notes, and see whether the high counts could be found again. From that time to the present those check experiments have been going on, and once a week a representative from the Department of Agriculture and a representative of the filtration plant visit every portion of the city, take the effluent water from the filter beds and draw water from the taps, take the samples to the laboratories, grow them upon the same media, and compare notes. The results have been uniformly the same. In addition to these special observations, daily tests have been made. The Secretary of Agriculture has also ordered a thorough inspection of the 900 dairy farms that supply milk to Washington. This inspection will include an extended series of bacteriological tests of the water supply of many of these farms. Up to the present time, many have been found in very bad sanitary condition, and the water supply of a large percentage of them has been found to be contaminated.

Another very interesting feature in relation to contamination and bacteriological count has happened in the last few days. Dr. Kober, in his paper in 1890, called attention to an apparent connection between an outbreak of typhoid fever in Cumberland, and an increase of the same disease in Washington a short time afterward. The Washington papers of October 28th, 1906, contain a report of a very serious outbreak in Cumberland, there being on that day more than 100 cases there. The speaker recently asked the authorities at the Agricultural Department, and also Mr. Longley,

as to the bacteriological count in the filtered water. Both reported that for the last two or three weeks the *Coli* have not increased. During the week ending October 28th, and up to the present time, they found that typhoid had diminished in Washington. It seems to be safe to say that, if Cumberland was the cause of the disease in 1889, filtration has saved Washington in 1906. Dr. Magruder.

A careful study of the Health Officer's reports from 1895 to the present time, with a view of trying to account for this condition, has brought out some very important points. In the report for 1895, Drs. Busey and Kober presented a very extended paper upon milk infection as the cause of disease, and there mentioned 131 epidemics of typhoid fever, besides scarlet fever and other diseases, following milk infection. A few years ago the speaker traced 16 cases of typhoid fever to one dairyman's farm. This dairyman had typhoid fever in his family, and the refuse from the patients was thrown out on the dump. The milk cans were washed in the same water that the family used, and the cans were contaminated by this water.

Investigations during the past summer into the influence of the milk supply upon typhoid fever have shown that 25 cases were traced to one dairy, 10 to another and 4 to a third.

One of the bacteriologists of the Agricultural Department gives a report of his examinations of the water supply of a certain dairyman who found 4 cases of typhoid fever in the families whom he supplied with milk. He had a little farm outside of the District, and being quite anxious about it, he engaged this bacteriologist to examine the spring supplying water for his farm. Almost 4 000 bacteria per c.c. were found, and most of these were of a sewage character. The vessels containing the milk were immersed in the spring, some of the water got mixed with the milk, and the patients had been poisoned by it.

St. Elizabeth's Hospital has about 2 600 inmates, and there has not been a case of typhoid fever among those who lived on the premises. There were 5 cases among employees, and those 5 cases arose from eating ice cream from a little saloon kept by a family in which there was typhoid fever. Every drop of milk used in that hospital is Pasteurized. The water supply is from very deep wells, and the hospital authorities seem to think that is the reason for the exemption.

Now, the speaker's contention, as to the Washington condition, is that the milk is really a much larger factor than has been admitted by either the Health Officer or by many writers on the condition of the city's health. His reason for saying this is also strengthened by the Health Officer's reports. There are 900 farms supplying milk to Washington. The Health Officer has about five officials for the inspection of these dairy farms, and, with this

Dr. Magruder. force, it is impossible to perform the proper service. Samples of the water supply are brought in, when the inspector thinks it is bad, and then it is only examined chemically. During 1903 and 1904 it was found that 46 farms had suspicious water. Samples were brought to town and examined; 26 showed by chemical analysis that they were unfit for use, and they were condemned. The others were considered good. In that case, out of 46, more than 50% was condemned as unfit for use.

During 1904 and 1905 there were 73 suspicious farms, and, specimens of the water having been examined, 25 were condemned as unfit for use, several of the remaining specimens were suspicious, and the others were considered good, by chemical analyses. There is no bacteriological laboratory under the District Government, and chemical tests alone are used for the water supply of dairy farms.

When it is considered that Dr. Kinyoun examined 6 farms and found the water of 4 of them contaminated with sewage, and that the water supply of another farm was found contaminated with sewage and reported by the bacteriologist from the Department of Agriculture, there are strong grounds for assuming that milk is one of the most important factors in regard to the increase of typhoid fever; and there is every reason to be thankful that the filtration plant has preserved the city from the contamination above.

The report of the Commission, appointed by the Surgeon-General, of the Public Health and Marine Hospital Service, approved by the Secretary of the Treasury, will probably be one of the most valuable reports that will be published in years. It will be ready in December, 1906.

There is no regulation in Washington governing the temperature at which milk should be delivered to the customers, nor are there any regulations requiring a bacteriological count. Already, during July and August, this Commission has found that much of the milk is delivered at a temperature exceeding 65° fahr.—some as high as 75°—and that the bacterial count, instead of being as low as half a million, which is the legal standard for Boston, ran up to the incredible numbers, 60, 70, and even 100 million bacteria per cc. More than 300 samples of milk, from 50 or 60 different dairies, were tested. This report will cover the analyses of every pump in Washington, every public pump, the daily tests of the tap water from the laboratory there, and tests of the tap water at each case of typhoid fever. The Commission has examined 800 cases of typhoid, and in each case the milk supply has been tested, and also the water supply, whether from a public pump, a private pump, or from the Potomac River. The Commission has also examined the fruit, the market vegetables, and things of that nature, and the report will be very exhaustive.

GEORGE W. FULLER, M. AM. SOC. C. E. (by letter).—This paper is of unusual interest, for several reasons. It is the first paper read before the Society on the construction and operation of filters of this type in America, if we exclude the valuable paper by Messrs. Knowles and Hyde on the special type of filter at Lawrence, Mass., since the presentation some seven years ago by Mr. Hazen of his very valuable paper on the Albany filters. Mr. G. W. Fuller.

The Washington filter plant is the largest now in operation in America, and, while it was designed, built and put in operation at a later date than some of the sand filters at Philadelphia, it is the first plant to be fully described in one paper showing the changes which have been made since the Albany plant was designed some 10 years ago. It is of further interest in that it deals with a much more turbid water than do the Albany filters, and that in some particulars advantage could be taken at Washington of certain important tests upon sand filters which were conducted at Cincinnati, Pittsburg and New Orleans, as well as upon the Potomac water itself at Washington.

As to the construction of these works, there are many instructive features, and it is well worth while for those interested in this line of work to study carefully the modifications which have been made at Washington as compared with the designs at Albany and elsewhere. One of the most instructive of these is the fact that sand filters can be constructed satisfactorily upon deep fills carefully prepared. The fact that some settlements occurred in the Washington filters at points where they were built upon an old fill, made before the land was purchased and surveyed by the Government, in no way detracts from the soundness of the experience above stated.

The manner of making the expansion joints in the various concrete walls is worthy of note, as it seems to be rather more effective than that followed at a number of places with which the writer is familiar.

The writer has taken an unusual interest in the very successful carrying out of the Washington filter project, due in part to his early connection with the project as a member of the engineering committee, appointed by the United States Senate Committee during the winter of 1900-01, to make recommendations as to the method of filtration to be adopted. There are many features of the project which are well worth detailed discussion, but the writer will confine himself to four points, as follows:

1.—Preparatory treatment of the river water.

2.—Preparation of the filter sand.

3.—Method of cleaning the sand filters.

4.—Relation of the filter plant to the prevalence of typhoid fever in Washington.

Mr. G. W. Fuller.

1.—Preparatory Treatment of River Water.—The Washington filters receive water from the Potomac River, which is passed through a set of three reservoirs, more or less irregular in shape, and holding about 8 days' supply. The displacement by the water flowing through these basins is not complete, and, in general terms, there is an effective period of about 4 days' subsidence.

The experiments conducted under the direction of the late Col. A. M. Miller, M. Am. Soc. C. E., were made on water which had not passed through the Washington City Reservoir, then under construction. The conditions of sedimentation, therefore, were not as favorable as they have been during the last year. Col. Miller recommended that mechanical filters be adopted, owing to the amount of clay in the settled water overtaxing the sand filters at intervals. This recommendation was strenuously opposed by the Medical Society and the Health Officer of Washington.

The Engineering Committee, Messrs. Hering, Fuller and Hazen, to whom the matter was referred by the United States Senate Committee on the District of Columbia, reviewed the local evidence and compared it with experiences available at Pittsburg, Cincinnati and elsewhere. The Committee recommended the adoption of sand filters aided by the use of coagulation during periods of marked turbidity in the water leaving the reservoirs.

It is noted in the paper that the coagulating department of the purification works at Washington has never been constructed. It is understood from the discussion by Dr. Kober that the official opposition of the Medical Society of Washington no longer obtains, but it is apparent from the discussion by Dr. Woodward that his opposition still continues.

As far as the writer knows, every engineer who is expert in matters of water filtration has advised the use of some adequate preparatory treatment which will reduce the turbidity of clay-bearing waters to a certain point so that sand filters may perform effectively and economically the duties for which they are intended.

It is not a new story that sand filters cannot deal effectively with very muddy water. The late James P. Kirkwood, Past-President, Am. Soc. C. E., satisfied himself upon that point, during his connection with the St. Louis Water-Works as Chief Engineer, more than 30 years ago. The same conclusion was reached by Charles Hermany, Past-President, Am. Soc. C. E., as a result of tests conducted at the Crescent Hill Reservoir, in Louisville, Ky., in 1883-84. Professor William Ripley Nichols was aware of these facts, as shown by his statements in his article on "The Filtration of Potable Waters."* The experiences of these gentlemen were obtained in testing sand filters during the days prior to the dawn of

* Massachusetts State Board of Health, Report for 1878, p. 154.

modern bacteriology. As Pittsburg and Cincinnati the effect of turbid waters in reducing the bacterial efficiency of filters during periods of prolonged freshets in the rivers was studied with care. At Cincinnati, the amount of turbidity, and the duration of the periods, in the Ohio River, afforded good opportunities for studying this point. The tests conducted there in 1898 for a period of about 10 months, under the charge of the writer, led to the conclusions stated in his report* to the late G. Bouscaren, M. Am. Soc. C. E., Chief Engineer, Cincinnati Water-Works, as follows: Mr. G. W. Fuller.

"While English filters are able to remove satisfactorily on an average about 125 parts of silt and clay of the unsubsided water, actual experience (see Chapter V, Section 1) shows that they can regularly handle suspended clay in subsided water in amounts ranging only as high as from 30 to 70 parts (depending largely upon the amount of the clay stored in the sand layer), and averaging about 50 parts per million. But it is true that for two or three days on short rises in the river, or at the beginning of long freshets, the retentive capacity of the sand layer allows of satisfactory results with the clay in the applied water considerably in excess of 70 parts. If this capacity is greatly overtaxed, however, the advantage is merely temporary, as the stored clay is washed out later, producing markedly turbid effluents.

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"It is necessary in this review to call attention to several abnormal characteristics of importance which appeared in the study of the local water, as follows:

"1. The comparative absence of organic matter in the water capable of forming gelatinous films around the sand grains at times when it was needed most. This was apparently due in part to the removal of the organic matter by attachment to sand and silt deposited in the river, and in part to further removal in the subsiding tank.

"2. The minuteness of the clay particles in the sudsided water, and difficulties in retaining them at and near the surface of the sand layer, especially in the comparative absence of organic matter adapted to increasing the retentive capacity of the sand."

* * * * * * * * * * *

"The system of purification under consideration (plain subsidence and sand filtration) was unsatisfactory along three lines, as follows:

"1. At times of prolonged freshets the turbidity of the filtered water was so great that it had the brownish appearance of the unfiltered water, frequently containing over 30 parts per million of suspended clay, an amount and of a character which makes it doubtful whether it could be removed by a second filtration.

"2. At times of freshets the penetration and retention of the clay within the main body of the sand layer would make the cost of

* "Report on the Investigations into the Purification of the Ohio River Water, for the Improved Water Supply of the City of Cincinnati, O.," pp. 382, 386 and 387.

Mr. G. W. Fuller.

scraping greater, and would apparently necessitate the reconstruction of the entire sand layer at much more frequent intervals than is normally the case in present practice with this type of filter.

"3. When the filtered water was turbid and brownish for some little time there was a marked tendency towards a diminution in the bacterial efficiency of the filters. During the long freshets frequently occurring during the winter the evidence points to the matter as one of serious significance."

The Mississippi River at New Orleans is much more turbid than the Ohio at Cincinnati. A set of tests on the use of sand filters under various conditions for the new river supply of New Orleans was conducted under the direction of George G. Earl, M. Am. Soc. C. E., Chief Engineer, with R. S. Weston, Assoc. M. Am. Soc. C. E., in immediate charge. These tests, under the oversight of the writer, as Consulting Engineer, continued for a period of about 10 months, beginning in the autumn of 1900. Some of the results were available at the time Messrs. Hering, Fuller and Hazen made their report upon the Washington project. The conclusions, as stated by Mr. Weston in his report to Mr. Earl,* are as follows:

"The Mississippi River water, after having been settled for 72 hours, is characterized by the absence of organic matter capable of forming gelatinous films on the sand, and by the presence of large numbers of very fine particles of clay. These clay particles themselves are somewhat gelatinous in their nature, and act within certain limits as a substitute for film-forming organic matter. These clay particles are sub-microscopic in size—many of them less than 0.00001 inch—0.00002 millimeter—and, when present in numbers exceeding a certain limit, penetrate deeply into the sand layer. Under these circumstances the operation of the filter, if long continued, may be seriously interfered with, and the removal of bacteria, as well as of the clay particles, may become unsatisfactory.

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"During the latter part of this investigation—summer weather—it was found that the filter produced an effluent of satisfactory character when the turbidity of the influent did not exceed 150 parts, silica standard, or 120 parts of suspended matter. With a well-ripened filter, constructed and operated under these conditions, it is very doubtful whether a satisfactory effluent could be obtained at all times with an unfiltered water as turbid as stated above. General experience certainly indicates that this would not always be the case.

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"It is judicious with this method of purification (coagulation basins and sand filters) during periods of muddy river water to apply sufficient coagulant to make the filter influent contain not much more suspended matter than could be satisfactorily removed from a

* "Report on Water Purification Investigation and on Plans Proposed for Sewerage and Water-Works Systems." Sewerage and Water Board, New Orleans, 1903, pp. 109, 111 and 126.

water which had not been treated with a coagulant. The available evidence indicates that it would be desirable to keep the turbidity of the effluent of the coagulating basin below about 35 parts per million (about 20 parts of suspended matter). * * *"

Mr. G. W. Fuller.

For explanations of the physical action of sand filters, see Mr. Hazen's paper "On Sedimentation,"* and the discussion on that paper by the writer.

The writer is just as firmly of the opinion to-day as he was when he signed the report of Messrs. Hering, Fuller and Hazen, in February, 1901, that the Washington sand filters at times require further preliminary treatment than is afforded the Potomac water by plain subsidence in the present system of reservoirs. In fact, the experiences of the last six years, obtained at New Orleans, Washington, and elsewhere, make the proposition clearer than it was before this style of filter plant was decided upon for Washington.

The reference to the use of coagulants by Dr. Woodward will no doubt be answered by the authors of the paper. In passing, it may be noted that Dr. Woodward's reference to the persons upon whom should fall the responsibility for proving the sanitary effect of coagulation seems to be turned about. It is a fact that coagulants for both public and private water supplies have been used for thousands of years in China, Japan, India, and Egypt, and that experience is by no means confined to the four million people in the United States who are regularly using water which is treated with coagulants. The use of coagulants for very muddy waters is not only imperative, but is now recognized as a standard procedure among engineers who give special attention to this branch of work. Whatever may be the objections to the use of coagulants in some localities where the waters are very soft, it is the judgment of the writer that the alleged sanitary danger in the use of coagulants, under such conditions as at Washington, is based upon fanciful and speculative reasons.

It is of interest to note that recently the use of coagulants in Europe has become more prevalent than it was a few years ago. Last winter, in visiting the filter plant of the Bremen (Germany) Water-Works,† the writer was interested to note that during muddy-water periods sulphate of alumina is being regularly used in connection with double filtration. It was stated that on an average this coagulant would be used probably about 50 days in a year. At Schiedam, Holland, in connection with double filtration, the muddy water of the River Maas is also regularly coagulated. At Gouda, Holland, coagulants are also used. In fact, it would not be difficult

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 59.

† "Filtration for Public Water Supplies, with Especial Reference to the Double Filtration Plant at Bremen, Germany," by Eugen Goetze, Chief Engineer, *Transactions*, Am. Soc. C. E., Vol. LIII, p. 210.

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to find quite a list of places where the value and importance of this method of treatment is recognized in practice. Perhaps the most important place not already mentioned is Antwerp, Belgium, where a sand filter plant is operated under the direction of Dr. Adolph Kemna, the well-known specialist in water-supply matters. A small amount of coagulant is constantly used at the Walheim works; and, during freshets, as much is used as is required to coagulate the water thoroughly.

It is interesting to note that, at both Bremen and Antwerp, this treatment following heavy rains is resorted to, notwithstanding the fact that the present coagulating basins afford a period of only about 14 hours' subsidence, and this is too short to allow all the hydrate to deposit before the water enters the filters.

It is very unfortunate, in the writer's opinion, that this controversy over the preliminary treatment of the Washington water has continued up to the present time, as indicated by Dr. Woodward's discussion. There is no doubt whatever that Mr. Hazen is thoroughly sound in his contention for the construction of preliminary works; and, in support of Mr. Hazen's position, the writer desires to go on record in a manner which is unmistakable.

This subject cannot be dismissed without stating that thus far the Washington filters have not received as much clay as the records of earlier years show that they will in the future, if reliance is placed solely on plain subsidence. During the past year the very muddy-water periods at Great Falls have been of quite short duration, and it has been possible on these occasions to shut the headgates to the aqueduct for several days at a time and draw from storage. This will not always be possible.

As to the question of giving a turbid water adequate preliminary treatment by other methods than by coagulation, the experiences at Philadelphia, and those outlined by Mr. Leisen at Wilmington, indicate that attention to such matters is required in the case of waters where the turbidity does not become too great. What the practicability is of roughing filters for the Washington water the writer would not attempt to say without further study.

If this long-continued controversy over Col. Miller's recommendation for the use of coagulants, at least for periods of very muddy water, is not soon settled, it would seem to be almost incumbent upon the authorities to have this matter fully and finally reported upon by a committee upon which there will be representation of all conflicting views.

2.—Preparation of the Filter Sand.—The feature of most interest to the writer during the construction of the Washington works was the very economical and effective manner in which the filters were charged with a layer of high-grade filter sand, taken

from banks where much clay was mixed with the sand. The use of the washing box described in the paper, an adaptation of certain principles used in washing mechanical filters, and, in fact, the entire arrangement for handling the sand, reflects great credit upon the engineers and the contractor. In the writer's opinion, this matter was handled better at Washington than at any other place with which he is familiar, either in America or abroad. Mr. G. W. Fuller.

The writer is fully in accord with the authors in the manner of handling the sand question as distinguished from the suggestions of Dr. Woodward, that the clay was too thoroughly removed. The wisdom of removing this clay will be apparent to any one who has studied the tests conducted by the writer at Cincinnati, where the storage of clay within the sand layer during periods of application of muddy water caused the sand layers to become seriously overloaded. In fact, there were instances at Cincinnati, following floods in the Ohio River, when sand filters received water which had been settled some two days or so, and was more turbid after passing through sand filters (with the sand layers overloaded with clay) than it was before reaching them. These are the conditions under which the bacterial efficiency of sand filters show well-defined deterioration as a result of inadequate preparation of the applied water.

3.—Method of Cleaning the Sand Filters.—This is the feature, connected with the construction and operation of sand filters, where the most marked advance has been made during the last few years, and where future improvements will be most conspicuous for some time to come. The Washington filters, designed in 1902, mark a substantial step in advance in the economy of cleaning sand layers, as shown by the cost data presented in the paper. Compared with the cost of cleaning sand filters some five years ago, the Washington data are very gratifying indeed.

Water-carriage, in transporting sand from one place to another, is utilized very advantageously, as compared with earlier plants at Albany, Lawrence and elsewhere; and the data in the tables showing the friction of sand and water through pipes of different sizes, with different velocities, etc., are important additions to our information upon this subject.

In regard to the use of the storage bins at Washington, and of teams for removing the cleaned sand from the bins to the filters, and through manholes in the vaulting, the writer is not in full accord. He recognizes the marked economy of this procedure, and that this economy may be somewhat greater in some cases than can be secured by other methods of demonstrated practicability. Nevertheless, there are disadvantages in the Washington method, especially in making the filtration works rather unsightly in appear-

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ance on account of the elevated storage bins and the frequent use of teams upon the sodded earth cover over the vaulting. Without much additional expense, these objections can be overcome.

It so happens that several sand filter projects with which the writer has been closely identified have been attended with such conditions that the Washington method would be impracticable. He does not wish to be understood as criticising seriously this feature in the design of the Washington filters, made four years ago, but he is clearly of the opinion that for many, if not most, projects, other and better methods are now available. A few words in general terms upon the present status of this element of sand filter designs may be of interest.

It has been demonstrated at New Haven, Conn., where a 4-acre sand filter plant, designed in the office of the writer in 1903, has been in operation some 8 months, that water-carriage may be successfully used in transporting the sand, not only from the portable ejector in the filter bed being scraped to the sand washers outside the filters, but also from the latter to the one or more filter beds which in turn are kept out of service for a time in order to receive clean sand. Experiences at New Haven show that this is practicable, and that the moving of the cleaned sand, from the point of discharge to its proper place within the filter, eliminates effectively the objection which was first raised to this method, namely, the separation of the sand into layers of graded sizes.

The general arrangement of the New Haven sand filter plant is shown in an article in *The Engineering Record*.* Data as to its operation have not yet been summarized, but the results are considered by the New Haven Water Company to be very satisfactory and economical.

Tracing the later history of water-carriage, it may be said that the designs made under the direction of W. B. Fuller, M. Am. Soc. C. E., for the Stormville filters, as proposed by the Burr-Hering-Freeman Commission on Additional Water Supply for New York City, carried the New Haven method still further. The plans for this proposed sand filter plant were published in the Report of the Commission in 1904. The Stormville design, however, was still further modified for the Pittsburg filters under the direction of Morris Knowles, M. Am. Soc. C. E.

It is said that at Pittsburg a traveling scraper and ejector will be used, by which the sand will be scraped and conveyed through piping leading to other ejector washers and finally to sand beds out of service, more or less after the fashion adopted at New Haven. It is further stated by the T. A. Gillespie Company, contractors for the Pittsburg filters, that, to meet Mr. Knowles' specifications, they

* Vol. XLIX, 1904, p. 270.

will install an arrangement manufactured by the Blaisdell Filtration Company, of San Francisco. This company manufactures still another device, which, it is claimed, effects a marked reduction in the cost of washing sand layers. This arrangement is accomplished by washing the sand in place, and it is said by several engineers to have been in successful use for several years at Yuma, Ariz. Mr. G. W. Fuller.

An arrangement more or less similar to that in use at Yuma is to be tried, among other devices, in the near future at the testing station now approaching completion at the Jerome Park Reservoir, New York City. This and other devices and methods are being studied in connection with the filtration works for the Croton supply by the Department of Water Supply, Gas and Electricity, under the direction of I. M. de Varona, M. Am. Soc. C. E., Chief Engineer. Mr. William B. Fuller, Filtration Engineer, is in immediate charge of this work, and Messrs. Rudolph Hering and the writer are Consulting Engineers.

4.—Relation of the Filter Plant to the Prevalence of Typhoid Fever in Washington.—In some ways the most important matters for discussion in connection with the Washington filter plant are the facts and theories associated with the rumor, which through the daily press has been widely published both in the United States and in Europe, that Washington has recently suffered from a severe typhoid fever epidemic, and that the responsibility of the epidemic is associated with the efficiency of the filter plant.

A thorough examination of the analytical data obtained at the laboratory of this filter plant convinces one trained in these matters that, with a water of the sanitary character of that of the Potomac River at Great Falls, there has been but small ground, if any, for associating the filtered water as a causative factor in the prevalence of any substantial amount of typhoid fever among the water consumers, unless at Washington there are conditions and factors not now suspected from local data or general experience elsewhere. However, this is a matter which it is not prudent to discuss in detail until all the evidence is available. It is understood that there will soon be an official report on the subject, issued by the Washington authorities, covering the work of a special commission under the leadership of Surgeon M. J. Rosenau of the Marine Hospital Service. The conclusions of the medical authorities would be highly desirable as a portion of the discussion of this paper, so that under one cover there will be a record of all the essential features associated with these excellent works.

In making exhaustive studies of the Washington situation, it appears that the improvement of the water due to sedimentation is entitled to considerable attention. It would be of interest if the writers would state the comparative degree of removal of turbidity

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and bacteria by subsidence during muddy-water periods. The experiences of the writer at Louisville and Cincinnati led him to believe that the percentage removal of turbidity and bacteria was about the same under the conditions studied for the Ohio River water.

The increased facilities for sedimentation, due to the completion of the Washington City Reservoir, were something of a factor, no doubt, in modifying the quality of the public water supply for a time prior to the operation of the filter plant, thus making an intermediate period as to the quality of the supply reaching the consumers. While it has been known for many years that subsidence is helpful in improving the quality of water, it cannot, of course, be compared in efficiency with filtration. Years ago, it was found at Lawrence, Mass., prior to the introduction of the city filter, that tap water in the City Hall showed a removal of about 93% of the bacteria in the river water, while the number in the tap water at the experiment station at the opposite side of the city from the distributing reservoir (2½ miles distant) showed a removal of 97 per cent. The reservoir had a capacity of about 2 weeks' average daily consumption. The uniformity in the improvement in the quality of the polluted Merrimac River water under these conditions, notwithstanding the fact that a severe typhoid fever epidemic then occurred, was quite marked, as will be noted from Table 23.*

TABLE 23.—MONTHLY AVERAGES OF THE NUMBER OF BACTERIA IN A CUBIC CENTIMETER OF WATER FROM DIFFERENT PARTS OF THE LAWRENCE WATER-WORKS.

1891.	Force main, pumping station.	Reservoir outlet.	City Hall tap.	Experiment station tap.
January	2 872	713	347	72
February	1 387	224	92	34
March	890	222	98	33
April	770	390	190	62
May	4 030	242	68	49
June	3 690	192	50	95
July	5 030	507	219	188
August	1 920	317	521	181
September	12 150	262	166	144
October	12 550	888	975	356
November	3 576	408	300	122
December	11 440	2 384	1 215	401
Averages	5 025	562	353	145

Water-works engineers are looking with great interest to what the medical authorities will have to say upon the question of

* Given by the writer on p. 632 of the 1891 Report of the Massachusetts State Board of Health.

"residual typhoid fever"; that is, the prevalence of that disease in communities known to have a substantially pure public water supply. The spread of this disease through the agency of flies is a factor which is of far more importance than was generally believed a few years ago, and even after the important reports made by the Commission appointed by the U. S. Department of War to deal with the prevalence of disease in the Army camps during the Spanish-American War. The significance of this matter was forced quite strongly on the attention of the writer while he was a member of the Commission dealing with the relation of the public water supply of Indianapolis to the prevalence of typhoid fever in that city during 1904. Mr. G. W. Fuller.

In this connection there is one thought which should be presented, namely, that there is increasing evidence to show that a uniform rate of typhoid fever in different sections of a city does not carry the weight that it formerly did in pointing toward the public water supply as the only common cause of infection. Typhoid fever infection may be transmitted by flies moving from place to place and following various vehicles for such a distance that it is no longer to be believed that the effect of unsanitary conditions in certain areas, as far as the public health is concerned, will necessarily be confined to the unsanitary area itself.

William B. Fuller, M. Am. Soc. C. E. (by letter).—The building and operation of the Washington filters adds another standard of comparison for such work, and it is interesting and instructive to note the fundamental changes in design from previous types. Mr. W. B. Fuller.

The irregular contour of the ground selected for the site would have caused many to seek another location, but its successful use demonstrates the fact that, with modern methods of excavation and embankment making, the roughness of the site need not necessarily rule it out.

As compared with other plants, concrete has excluded almost all other materials as a material of construction, and the growing confidence in its use is exemplified by the decrease of 11% in the masonry sections at Washington over the Albany sections designed only five years previously.

Mr. Hazen's design for decreasing the size of the main collector, without producing unequal rates of filtration over the sand area, by introducing compensation orifices into each of the lateral drains, is a very novel and theoretically perfect method, and its operation in practical use will be watched with interest. It is to be hoped that the authors will add to this information by giving a table showing the area of each orifice at each lateral, and, as the solution of such problems is very complicated, an example of the methods of computation would be of interest.

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The use of three grades of gravel, with a total thickness of 12 in., as at Albany, while undoubtedly serving the purpose, requires a rather wide difference in size between the sand and the finest grade of gravel. The gravel is specified as passing a $\frac{3}{8}$-in. mesh and being coarser than ordinary sand, therefore it will have an effective size of about 3 mm., while the sand which it supports has an effective size of about 0.30 mm. The fact that the sand does not work down into gravel of this size must be due to the fact that both the gravel and sand are somewhat graded, as (diameters being only relative) one would hardly expect a layer of stones, each, say, 10 in. in diameter, to support a layer of stones each only 1 in. in diameter, without many of such stones working downward through the interstices of the larger ones. The writer prefers, at a little extra cost, to add an intermediate size of stone at this place, and make the total depth of the gravel 16 in.

It is interesting to note the improvements in scraping, washing and transporting sand, and the Washington plant indicates the successful application of improvements in this direction and the cutting down of the maintenance expenses in a marked degree. The sand washer is a decided improvement over earlier types, but the writer believes that the efficiency of the ejector can be increased still further; the thought being that it should be separated into two parts: First, that which introduces the sand into the pipe; and second, the dilution of the sand to proper consistency and its transportation. To this end he would suggest an inspirator, just ahead of the ejector, which will produce zero pressure in front of the ejector, allowing all its energy to be used in entering sand, the inspirator providing the necessary extra water and pressure for transportation.

The use of the Venturi meter for measuring rates is to be commended as producing an accurate reading with only a small total loss of head, and doing away with the troubles of open and submerged orifices and weirs. It is to be hoped that a reliable automatic device for regulating the valve at the outlet of the Venturi, so as to keep an even rate, will at some future time be available, as a reliable automatic regulation is to be preferred to hand regulation.

The building of a clear-water reservoir, so as to allow the water to rise to the full height of the water in the filters, would hardly seem advisable except under unusual conditions of operation, such as a very small amount of available loss of head. The level of the clear-water reservoir must be kept about 5 ft. below that of the filters during operation, and to build higher than this involves extra cost in walls and piers hardly warranted by the temporary gain in storage.

The automatic closing down of the filters by the backing up of

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the water from the clear-water reservoir would also appear to be undesirable under ordinary conditions, as those filters having the largest loss of head would be closed down first, allowing the loss of head on the newer filters to go on accumulating, and if such fluctuations occurred often, it would result in the entire plant going out of service at the same time. The writer prefers to maintain the high-water line of the clear-water reservoir at a predetermined elevation below that of the filters, and, when this high-water line is reached, to close down the newly cleaned filters by hand in the reverse order of cleaning. This allows the filters longest in use to complete the run and go out of service instead of being held back.

It would be very instructive if the authors would say a little more about the hose used for transporting sand, its life, and the kind found most effective for resisting wear.

The use of sand bins is a novel idea, and is a decided improvement over the older method of storing sand openly in the courts. It gives a cleaner and more business-like look to the entire plant. The writer, however, would prefer to do away with the courts and bins entirely, and utilize this space for extra filter beds, replacing the washed sand directly in a spare filter bed instead of in the bins, thus saving the subsequent removal and placing. This method is in successful use at the filter plant at New Haven, Conn., designed by Messrs. Hering and Fuller; and the well-grassed lawn over the filters, the absence of sand piles, open courts and machinery, make the plant very noticeable in its contrast of cleanliness.

Not the least of the benefits to be derived from keeping the sand in the beds at all times and carrying on the operation in covered galleries instead of open courts is the greater opportunities allowed for artistic treatment of the filter site by broad landscape gardening effects, with green stretches of lawn interspersed with formal gardens, hedges and artistic shelter and operating buildings, the whole making a modern public garden and park. Such expenditures are greatly appreciated by the average citizen, and instil confidence in the proper operation of the plant—which no amount of statistics would be able to accomplish—and it is confidently predicted that in future plants money will be counted as well spent when applied to the artistic as well as the commercial side.

The great amount of study which has been given to the operation of filters in the past five years has resulted, as is shown by the Washington results, in a marked decrease in operating expenses; but, as these have usually been accompanied by higher installation expenses, it has become a question, in some of the later plants, whether the first and simplest methods were not really less expensive in the long run. Numerous attempts have been made to reduce such installation expenses, and gratifying success has been attained at

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Yuma, Ariz., and Hempstead, Long Island; and it is confidently predicted that further improvements in washing sand *in situ* will yet further reduce both first cost and operating expenses.

This aspect of the case has been very carefully considered by the writer, in preparing the plans for the proposed new filters for the Croton water supply of New York City, at Jerome Park Reservoir.

Mr. Horton.

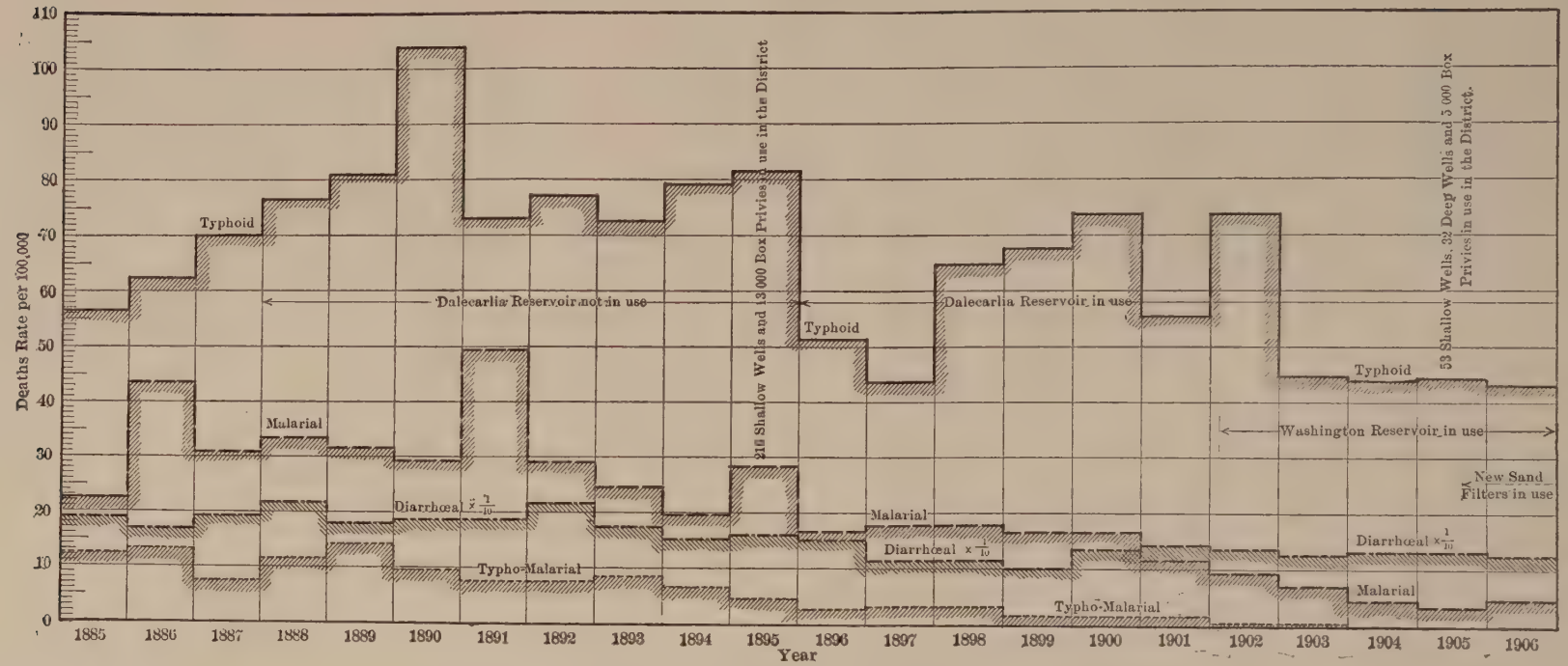
THEODORE HORTON, M. AM. SOC. C. E. (by letter).—The writer has read this valuable paper with much interest, and has been impressed with the description of the plant, not only from the standpoint of design and construction, but also with respect to the high efficiencies attained by it in the removal of bacteria from the Potomac River supply. In view of these high efficiencies the writer felt somewhat surprised, if not disappointed, on learning that, notwithstanding the installation of so complete a plant, and its continuous operation since October, 1905, there had been no reduction of the typhoid fever rate in the District of Columbia, as had been confidently expected.

So varied and sensational were many of the reports which had been circulated freely throughout the country in regard to the situation at Washington that the writer was engaged to investigate and report upon the prevalence and causation of typhoid fever in the District of Columbia, and in view of the close relation of the subject to the purification of the water supply of Washington, he feels justified in presenting the substance of his report in the hope that further discussion of both subjects may be brought out.

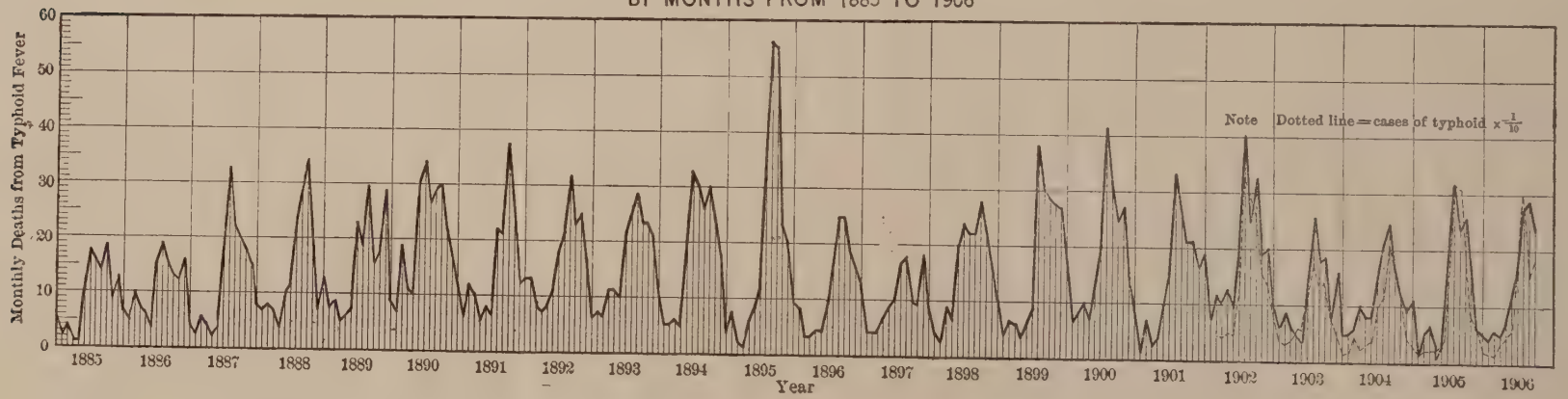
Although the time available for undertaking this work was inadequate for making as thorough a study of it as was desirable, the writer spent portions of three weeks, mostly on the ground, acquainting himself with local conditions and gathering vital statistics and other information which might throw light upon this apparently perplexing situation.

Statistics of typhoid mortality for some twenty years or more, compiled from records of the District Health Officer, were courteously placed at the writer's disposal, and were carefully reviewed, tabulated and charted. Records of the operation of the new filters, from the time they were put in service, were also given to him, and were studied carefully. Still other statistics, relating to the amount of pollution upon the Potomac River water-shed, which, fortunately, had been gathered recently by the United States Geological Survey, were studied. Information of a specific nature, relating to wells and privies was secured, and also relating to dairies, shell-fish, market and garden truck, etc. Finally, a personal investigation was made of the causation of a few actual cases of typhoid fever in the District.

MORTALITIES PER 100 000 PER YEAR FROM TYPHOID, TYPHO-MALARIAL, MALARIAL AND DIARRHŒAL DISEASES IN THE DISTRICT OF COLUMBIA FROM 1885 TO 1906



DEATHS FROM TYPHOID FEVER IN THE DISTRICT OF COLUMBIA BY MONTHS FROM 1885 TO 1906



Mr. Horton.

The question of first importance was to determine somewhat closely from official records the prevalence of typhoid during 1906, in order that it might be compared with typhoid statistics of past years and with similar statistics in neighboring localities or cities. Fortunately, and with much credit to the management of the present District Health Office, under Dr. William C. Woodward, complete records for more than 20 years have been compiled, showing the mortality from typhoid and other allied diseases classified as typho-malarial, malarial and diarrhœal. In addition to these mortality records, morbidity records of typhoid have been kept for the past 4 years.

The history of these four diseases in the District of Columbia is shown graphically, and in considerable detail, from January, 1885, to October, 1906, by two diagrams on Plate XL. On the upper diagram are plotted curves showing the yearly mortality from these diseases. The curves represent mortalities per 100 000, except that for diarrhœal diseases the ordinates represent mortalities per 10 000, or one-tenth of the mortality per 100 000. A few explanatory notes, relating to wells, privies and other features which may show an association with these curves, are also recorded chronologically upon the diagram, and will be referred to in later statements.

On the lower diagram on Plate XL are plotted the deaths from typhoid in the District for each month beginning with January, 1885, and the reported cases of the disease for the past 4 years. For convenience, and for the sake of closer comparison, this morbidity curve was plotted with ordinates representing one-tenth of the actual morbidity; and in this respect shows roughly the fatality from typhoid—below or in excess of 10%—during these years.

Keeping this historical picture in view, the changes in the mortality rate from these four diseases, from year to year and from season to season, will be traced carefully, in order to obtain a clear idea of these variations and discover, if possible, their significance when compared with the typhoid rate during 1906.

Referring first to the upper diagram on Plate XL, and comparing the yearly mortality rate of the four diseases, it is observed that, in general, all remain about the same, or show a gradual increase, from 1885 to 1890. From this date up to the present time, there is a general though irregular decline in all.

Considering them in more detail, it is seen that the curve of malarial diseases rises gradually from 22 in 1885 to a maximum of 49 in 1891, and then falls to about 4 in 1903, after which it does not change materially. The typho-malarial curve shows a similar but more striking history, as it rises from 12 in 1885 to a maximum of 14 in 1889, and then steadily decreases to zero in 1904, after which it fails to be recorded, indicating clearly the ill-defined nature of

Mr. Horton.

the disease and the probability that, under improved methods of differential diagnosis in later years, it has been classified either as typhoid or as malarial fever. The curve for diarrhœal diseases does not show this fluctuation in so marked a degree, probably owing to the greater number of deaths recorded, which would tend to keep the average more uniform; the general decline after 1892, however, is apparent.

The curve for typhoid, though following in a similar way the variations of the other curves, is strikingly more erratic. In general, however, it rises from 56 in 1885 to a maximum of 104 in 1890, and then declines to about 45 in 1903, after which it remains nearly stationary. The irregularities, however, are quite marked. The mortality rate runs quite uniformly for periods of a few years, for instance, from 1891 to 1895, and from 1903 to 1906, while, for the intervals between these periods, the curve is so irregular that possibly one might be led to consider the disease epidemic if the records of one year were compared with those of only the preceding or succeeding one.

Referring to the lower diagram on Plate XL, which shows the seasonal distribution of typhoid, even more striking features are found. Taking, for instance, any one year, a very marked fluctuation is found during the different months. Beginning about May, the curve rises gradually, reaches a maximum in August or September—at which time the mortality rate is about four times as great as during the spring months—and then declines steadily until the following January, remaining about stationary until the following May.

The relation of the seasonal variation of the typhoid mortality with the temperature has been studied carefully by Liebermeister in Germany, and, for different localities, by Sedgwick and Winslow in America, and so important is this relation to the problem at Washington that Fig. 27 has been prepared to show it.

This diagram shows the average mortality from typhoid for each successive month of the year for the past 20 years, and the average of the mean monthly temperatures for the same period. For additional comparisons, the diagram also shows the average of the monthly relative humidities and rainfall for this period. These four curves, then, show any relation which may exist between the seasonal occurrence of typhoid and the seasonal changes in meteorological conditions. Considered broadly, there is a marked similarity between the curves of typhoid mortality, temperature and rainfall. With the curve of relative humidity this similarity is less marked, though apparent. It is interesting, however, to note that the typhoid curve is distinctly more nearly synchronous with the relative humidity curve than with the curves of temperature and rain-

COMPARISON OF SEASONAL MORTALITY FROM TYPHOID WITH SEASONAL CHANGES IN TEMPERATURE, RELATIVE HUMIDITY, AND RAINFALL, FROM 1885 TO 1906.

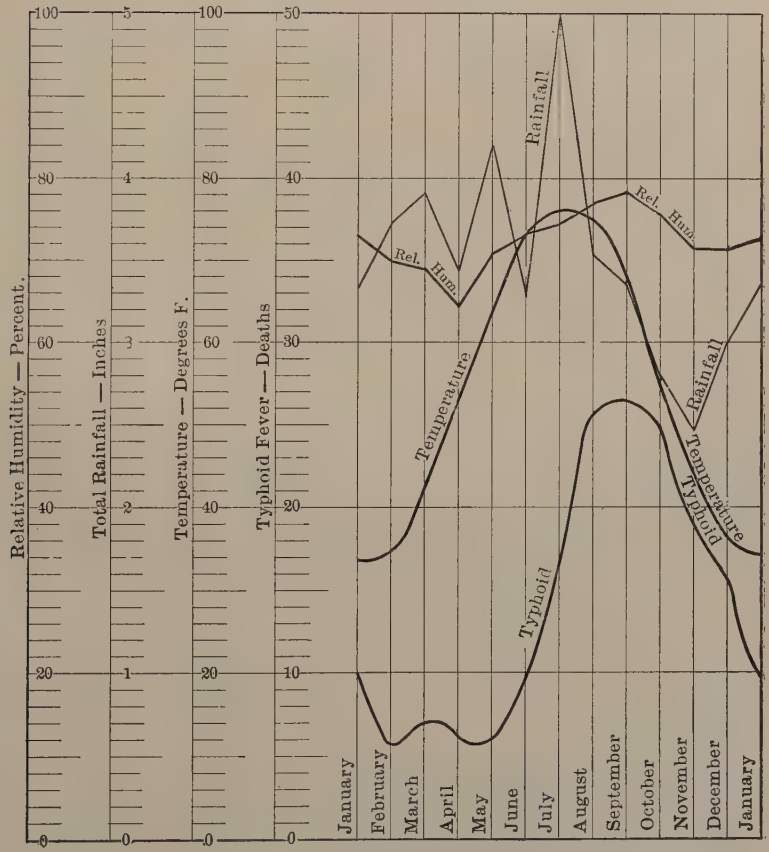


FIG. 27.

Mr. Horton. fall, which would seem to indicate that any relation which may exist is one which affects the fatality more than the incidence of the disease.

The typhoid and temperature curves display a remarkable parallelism. The similarity between them is striking, notwithstanding the fact that occasional outbreaks of typhoid have occurred which would tend to destroy it. If allowance be made for slight irregularities in the typhoid curve, and also for the necessary time which must elapse for infection and incubation of the disease, as well as for the fever period before death, it will be seen that the two curves are almost identical. An important fact is that the parallelism of the mortality curve of typhoid with the seasonal curve of temperature for the District of Columbia had already been pointed out and commented upon by Professors Sedgwick and Winslow, of the Massachusetts Institute of Technology, in their valuable paper upon "The Seasonal Prevalence of Typhoid Fever."* The studies of these authors showed that the character of the curves of typhoid mortality for the District of Columbia, for the period studied by them, were similar to those of Boston, New York, London and other cities, which, however different in the amount of typhoid present, are all alike in exemption from epidemics of water-borne typhoid. The writer finds that the conclusions of these authors for a period of 10 years are not less true for the much longer period which he has investigated.

Returning to a more detailed discussion of the mortality from typhoid, as shown by the lower diagram on Plate XL, there appears the important fact that, in spite of the operation during the present year of an elaborate and costly purification plant, built for the express purpose of reducing the typhoid mortality, no such reduction as was expected—in comparison with the two years just preceding—has taken place. In view of this fact, it is not surprising that much disappointment has been felt, nor that some have gone so far as to pronounce the plant a failure. Before coming to such a conclusion, however, it is obviously necessary to examine carefully all the available data bearing upon the prevalence of typhoid, not only in the District of Columbia, but in similar communities elsewhere.

The total of typhoid fever for any community is always the sum of a number of component factors, such as polluted water, polluted oysters, polluted vegetables and other polluted foods. There is also usually a considerable amount derived from direct contagion, sometimes called "contact" or "secondary infection." These various factors, in relation to the local conditions studied and observed in the District of Columbia will now be considered under a series of corresponding headings.

* Memoirs, American Academy of Arts and Sciences, Vol. XII, No. V, August, 1902.

Mr. Horton.

1.—Distribution and Classification of Typhoid Fever Cases and Deaths.—In general it was found that the cases of typhoid were quite uniformly distributed over the entire District—that is, roughly in proportion to the density of population. With few exceptions, and making allowance for cases yet to be recorded during the remainder of 1906, it appears that in 1905 and 1906 typhoid was prevalent in every section of the District, and that in no particular sections could the disease be considered distinctly more prevalent than in others. This fact of uniform distribution is of considerable importance in throwing light upon the possibility of any particular foci of infection. It points, in fact, rather to a general infection from either the public water supply, the continued general use of wells and box privies, or the more complex general causes now classed under the head of prosodemic typhoid fever.

Another classification was made by taking 200 cases, as they appear in order on the records, and determining what percentage of them was imported and what percentages were in families of the lower, poorer class, the middle class, and the upper or more wealthy class. This classification indicated that, of the 200 cases, approximately 12% was imported, 60% was among the lower, poorer class, 30% among the middle class and only 3% among the upper or wealthy class. This, again, would seem to point to wells, privies and other causes of a prosodemic nature.

Another classification was made between the white and colored population. The colored population is about 30% of the total population of the District, whereas the fatality from typhoid among the colored race is nearly double that among the white race, being in 1905, 28.7% for the negroes and 15.1% for the whites. This is a very important fact, for it gives a much higher typhoid death rate for the District than would exist if the population were mostly white, as in cities situated farther north. This classification by races, however, has no particular bearing upon the relative prevalence of typhoid for 1906, because the conditions for this year, as far as the writer could learn, are not materially different from those of recent years.

2.—Public Water Supply.—In view of the uniform distribution of typhoid throughout the District, and the apparent failure of the filtration plant to give the expected reduction of the typhoid rate, to say nothing of the specific imputations frequently heard regarding the operation of the plant or its value as a means of reducing typhoid, the question of the influence of the public water supply was studied carefully.

The new filtration plant was completed and put in service in August, 1905, but it was not until October that unfiltered water was entirely excluded. From the date the filters were put in service,

Mr. Horton. very complete and elaborate records of its operation have been kept, and these records, to a surprising degree, show the efficient work of the filters in removing bacteria from the raw water. In order to show this clearly, the average bacterial efficiency of the filters from January to October, 1906, is presented in Table 24. The table gives the average number of bacteria per cubic centimeter from January to October, 1906, in the raw Potomac River water at the intake at Great Falls; after passing through each of the three reservoirs—two natural and one artificial—located along the aqueduct between the intake and the filters; and the bacteria in the effluent delivered into the city mains. The efficiencies are expressed in two ways: first, showing separately the percentage removal due to passing through each of the three reservoirs and the filters; and, second, showing the total percentage removal as compared with the bacteria in the raw Potomac River water.

TABLE 24.—AVERAGE NUMBER OF BACTERIA PER CUBIC CENTIMETER IN THE RAW POTOMAC RIVER WATER AT GREAT FALLS, AND AFTER PASSING THROUGH THE DALECARLIA, THE GEORGETOWN, AND THE WASHINGTON CITY RESERVOIRS, AND THE FILTER BEDS; WITH CORRESPONDING EFFICIENCIES OF BACTERIAL REMOVAL DUE TO THESE FOUR FACTORS, FROM JANUARY TO OCTOBER, 1906.

	Bacteria per cubic centimeter. Average of months January to October, 1906.	Percentage removal due to each separate reservoir, etc.	Percentage removal compared with raw Potomac River water.
Great Falls	4 900		
Outlet to Dalecarlia Reservoir	1 480	70.0	70
Outlet to Georgetown Reservoir	700	52	86
Outlet to Washington City Reservoir (as it passes upon the filters)	300	57	93
Effluent from filters	18	94	99.6

Two very striking facts are made apparent by Table 24. One is that the Potomac water, after passing through the three reservoirs, where it undergoes a high degree of subsidence, and, before passing through the filters, shows a reduction of about 93% in the number of bacteria in the raw water. An interesting confirmation of this purification is readily obtained by comparing, on the upper diagram on Plate XL, the typhoid rates during the periods when the Dalecarlia and Washington City Reservoirs were in use, and when they were not in use. The periods indicated in the diagram are only approximate, but they show the effect of the purification by subsidence upon the general typhoid death rate of the District.

Mr. Horton.

The other fact is that the efficiency of the filters in reducing the bacteria is very high, being 94% of the applied water, and 99.6% of the raw water. In the latter case the removal is practically complete, leaving only 18 bacteria per c. c. in the filtered water delivered to the city. It is needless to say that such completeness of removal has been rarely if ever maintained elsewhere.

Considerable comment has been heard in regard to finding the sewage bacterium, *Bacillus Coli,* in the filtered water; but, as such bacteria are in the unfiltered water, it would be folly to expect that without perfect removal they would not be present in small numbers in the effluent. Fortunately, *Coli* tests are made regularly at the filter plant, and have also been made by the Health Department and the United States Public Health and Marine Hospital Service, which department is at the present time investigating this subject officially. These bacterial analyses, especially those made regularly at the filter plant, show that *Colon bacilli* are reduced in passing through the filter in about the same proportion as the other bacteria —in fact, these *Coli* analyses check clearly, in an independent way, the efficiency of the filters as determined by ordinary plate analyses.

In brief, so completely do these records show a high degree of efficiency of the filters and high purity of the water now supplied to the District that they may be taken as very strong evidence that the cause of the present typhoid cannot be attributed, in any appreciable degree, to the quality of the water supply.

3.—Milk Supply.—The milk distributed in the city comes from some 800 dairies, mostly small ones, situated in the rural districts of Maryland and Virginia. These small dairies ship the milk to some 200 dealers in Washington, who bottle it and distribute it locally throughout the District. Although a separation of the milk is made after it is received by the dealers, it is, in general, more or less mixed before it is bottled and distributed. The sterilization of bottles is practiced to a degree, but not generally.

A rather careful control of the milk trade is kept by the District Health Department, which has a number of men constantly making inspections and reporting the conditions existing at the dairies. Any abnormal occurrence of infectious diseases upon any one route is at once investigated, and effective steps are taken to prevent further distribution of such milk until all danger is past.

That the milk supply exerts an appreciable influence upon the typhoid rate in Washington, notwithstanding these facts, is made evident by the circumstance that outbreaks are observed occasionally —averaging from two to three each year. There have been three such outbreaks during the past year, including one during the writer's recent visit. In fact, milk unquestionably plays an important role in the relatively high typhoid rate of the District, and

Mr. Horton. there are undoubtedly many cases of typhoid attributable to this cause which are not shown clearly by mere inspection and records of reported cases. This subject of the influence of the milk supply in the causation of typhoid has been studied very carefully by George M. Kober, M. D., during the past ten years, and, as a result of his investigation, he has estimated that fully 15% of the cases of typhoid in the District may be attributed to milk infection.

4.—Oyster Supply.—Frequent inquiries indicate that more than 95% of the people of the District eat shell-fish in the proper season, and that a considerable number eat them out of season. The volume of the trade and the extent to which oysters and clams are eaten at different seasons, for the year 1905, are shown in Table 25.

TABLE 25.—VOLUME OF OYSTERS AND CLAMS SHIPPED INTO THE DISTRICT OF COLUMBIA AND CONSUMED IN 1905.

Month.	Oysters imported, bushels.	Clams imported, number.*	Cases of typhoid fever reported.
January	42 400	265 000	31
February	14 300	230 000	10
March	27 700	540 000	24
April	16 800	730 000	24
May	4 700	1 265 000	28
June	2 000	500 000	40
July	2 100	860 000	133
August	1 000	385 000	314
September	19 300	240 000	212
October	36 100	370 000	150
November	54 800	265 000	80
December	53 700	253 000	52

* There are about 300 clams to one bushel.

About 75% of the oysters comes from the beds along the lower reaches of the sewage-polluted Potomac River, although none are taken from within 70 miles below Washington. The remainder come from Chesapeake Bay. Any "fattening" is done before the oysters are shipped to Washington, as none are fattened in the Potomac River near Washington. The clams are taken principally from the salt waters of the lower Chesapeake near Norfolk. The fact that large quantities of oysters and clams are consumed in the District, and that some are consumed out of season, and also the fact that some come from sewage-polluted beds, would indicate that shell-fish contribute an appreciable share to the typhoid in the District.

5.—Vegetables, Fruits and Berries.—The quantities of these foods consumed by the public in a raw and unsterilized state are unquestionably relatively large. This is especially true of fruit and

berries during the summer. These are brought in from the neighboring rural districts where, as described later, the prevalence of typhoid and malarial diseases is greater than in Washington. It is admitted that conditions of truck-farming in this locality, as elsewhere in the South, are often unsanitary. The vegetables and berries are of necessity gathered by hands which are at times infected, and, if washed at all, are often washed in sewage-polluted waters; fertilizing is frequently done with human as well as cattle manure; and the products, on reaching the markets, are exposed for sale in open stalls and handled in a manner favoring still further infection. The writer inspected carefully a few of these markets, and, whereas the better ones were attractive and as sanitary as markets are generally kept, he found conditions there, when taken together with the others just mentioned, such as to justify the conclusion that the "garden truck" as collected, exposed for sale, and consumed, without sterilization by cooking, is responsible for a considerable share of the typhoid in the District. Mr. Horton.

6.—Public Wells.—In 1895 there were 216 shallow wells distributed in various sections in the District, but mostly within the city limits. In 1902 there were only 62 of these shallow wells, but in this year the city caused some 40 deep wells to be sunk, hoping to secure better public well supplies and do away with the shallow wells. In 1906, 53 shallow wells and 32 deep wells were still in use.

These wells have been attacked frequently by the more zealous local sanitarians as the principal cause of typhoid. This view no doubt has been justified at times and in particular cases. Although chemical analyses indicate that the water reaching most of the wells has at some time received sewage pollution, both chemical and bacterial analyses, made by the District Health Office and also by the chemists and biologists attached to the filtration plant, show that the water has been thoroughly purified before reaching them. Thus, of the 19 deep wells now in use within the city limits, only one showed the presence of *Coli;* and of the 41 shallow wells in the city only 12 showed the presence of *Coli,* and of these 12 only 4 gave total bacteria in excess of 100.

It is thus evident that the wells now in use can hardly be considered an important factor either in the general occurrence of typhoid in the District or in its continued prevalence during the year. The wells were found to be fairly uniformly distributed throughout the District, indicating that there can be no specific prevalence of the disease attributable to any particular well or wells; and, further, considering their degree of purity they can hardly be considered an important factor in the general typhoid mortality. That they should be abandoned, however, is self-evident, for there is always a chance for surface infection, and the District now has a much purer and safer water supply.

Mr. Horton.

7.—Box Privies.—The influence of box privies on the typhoid mortality of the District has always been, and continues to be, a subject of much speculative thought and opinion. It has been exploited occasionally as the principal cause of the continued presence of the disease, and, in the writer's judgment, this opinion has been in a measure justified.

These box privies are constructed of wood upon a masonry base, and are entirely above ground. Though admittedly water-tight when first constructed, their life is short, and they soon become dilapidated and leaky. Their exposed condition and especially the leakage from them flowing upon or over the surface of the ground serve as a means for the spread of infection either by direct contact or indirectly through the agency of flies.

It is estimated that in 1895 there were some 12 000 to 14 000 box privies in use in the District. Although, according to official records, there has been an abandonment of some 5 000, mostly within the city limits, the records do not show how many new ones have been constructed during the same period. It would probably be fair to assume that some 7 000 or 8 000 are still in existence, of which probably some 4 000 or 5 000 are within the city limits.

These privies are very generally distributed over the city and District, and, although more concentrated in some sections than in others, there is no clear indication of a greater prevalence of typhoid in the sections where they are most numerous; yet, when it is considered that infection from them is very often indirect, this fact is not important, as many of these unsanitary foci of general infection exist, and must of necessity have an appreciable influence in the general spread of typhoid.

8.—Prevalence of Typhoid in Other Localities of the South.—Admitting that the mortality from typhoid in the District of Columbia has as yet shown no considerable reduction during the present year when compared with the last two years, and in order to throw light upon the important question as to whether typhoid in this section of the country, outside of the District of Columbia, might not be more prevalent this year, and consequently the mortality within the District relatively less, an attempt was made to determine the relative prevalence of typhoid in neighboring localities. Such evidence, if it could be determined accurately, would indicate whether the typhoid mortality in the District was only apparently the same this year as compared with previous years, or whether there was relatively a decrease or an increase over former years.

Although at this season health records for the current year are not generally available, the writer communicated with health officials in Cumberland, Baltimore, Richmond and Atlanta, and was per-

sonally familiar with typhoid conditions at Norfolk. Actual statistics were available for Atlanta and Norfolk, and these show that, at Atlanta, the cases of typhoid during July, August and September, 1906, were more than the sum of the cases in the corresponding months of 1904 and 1905 combined. In Norfolk, the mortality rate for the last summer months was about twice as great as during the corresponding months of any one of the past three years. Dr. Wiley, of Cumberland, states that there is as much typhoid this year in Cumberland and surrounding towns as during the last two years combined. Dr. Fulton, of the State Board of Health of Maryland, states in a general way that in Maryland the prevalence of typhoid this year is from one-quarter to one-half greater than in any of the last few years. Dr. Levy, of Richmond, states that typhoid in that city is no greater than in previous years, but gives no statistics of the relative prevalence of the disease outside that city. Mr. Horton.

Mr. M. O. Leighton, Chief of the Hydrographic Division of the United States Geological Survey, states that in the rural districts of the Potomac River water-shed information gathered by his Department showed that typhoid was very prevalent during the past season over the entire area. These districts are inhabited principally by the poorer classes of the colored race, and, owing to poverty and the lack of medical attendance, the disease is not only more prevalent, but it is difficult to obtain reliable information except by a house-to-house canvass. Although this investigation furnished no statistical information throwing light upon the occurrence of typhoid as compared with previous years, the general impression is that it is especially prevalent.

Evidence obtained in this general manner is lacking in statistical value, yet, when considered as a whole, and in view of its responsible sources, it is significant as indicating that there has been a greater prevalence of typhoid outside the District of Columbia this year than in the past two years, and that the prevalence of the disease in the District this year may be considered relatively less than in the past two years. In fact, it suggests strongly that the additional purification of the public water supply resulting from the operation of the new filtration plant has already shown to some extent the expected decrease in the typhoid.

9.—Results of Personal Investigation of a few Cases of Typhoid in the District of Columbia.—For the purpose of getting a more definite idea of the causation of the typhoid which prevails in the District at present, as well as to obtain some idea as to the proportion of cases which might have been imported, or be due to secondary infection, the writer made a house-to-house canvass of a number of cases where recovery occurred. These were taken from the

Mr. Horton. records somewhat at random, but in such a manner as to include cases among the poorer class, the middle class and the more wealthy class of citizens. As the information received throws considerable light upon these points, it is presented in detail:

Case I.—Middle class, female, age 40, boarding-house keeper, sickness in July, treated in hospital, case normal. Public water supply, local milk dealer; no oysters, and few raw vegetables, eaten prior to illness. Surroundings sanitary. Visited family with case of typhoid at intervals for some weeks prior to sickness. Two convalescents from typhoid kept as boarders, prior to sickness. Case apparently secondary.

Case II.—Middle class, male, age 16, sickness in July. Absent from home in Western Maryland for five weeks; during that time roomed with companion who contracted typhoid and died. Case apparently imported.

Case III.—Upper class, girl, age about 18, date of sickness, July 10th. During six weeks prior to illness had visited two weeks at Roanoke and ten days at Rockville, Md. Five other cases occurred in same family, in following order: July 16th, July 18th, July 22d, July 25th, and August 6th; all recovered. Case apparently imported.

Case IV.—Upper class, male, age 40, banker, date of sickness, August 1st. Not out of city prior to sickness. City water, local milk dealer, surroundings sanitary. Did not eat raw vegetables or oysters. No association with any other typhoid cases. Cause not clear.

Case V.—Upper class, female, age about 18, date of sickness, July 15th. Society girl who visited many sections of the city. Ate a great deal of lettuce and salads, little milk, and no oysters prior to illness. During one visit drank water noticeably offensive. Drank water from public hydrant when playing tennis. Surroundings sanitary. Cause not clear.

Case VI.—Middle class, male, age 40, date of illness, July 15th. Made trip by boat to Baltimore for few days in latter part of June. Fond of fish, but did not eat oysters at this season. Public water supply, except on boat at Baltimore; milk from local dealer, except on trip. No association with other cases of typhoid fever. Cause not clear.

Case VII.—Lower class, German, age 8, date of illness, August 15th. Surroundings very unsanitary—small baking and grocery shop—"Provisions bought where obtained the cheapest"—ate freely of provisions and drank freely from two public wells. Not associated with other cases of typhoid fever. Cause not clear.

Case VIII.—Lower class, colored, male, age 8, date of illness, July 25th. Visited at Rockville, Md., for two weeks prior to illness.

Surroundings sanitary, except wet cellar. Mother said infection was received at Rockville (note that Case III visited Rockville). Case probably imported. Mr. Horton.

Case IX.—Lower class, colored, male, age 14, date of illness, September 10th. On visit to Culpepper, Va., for three weeks prior to illness, and was brought home sick. Case imported.

Time was not available to make a more thorough or extended canvass, which is much to be regretted in view of the large percentage of imported cases among so few investigated. Enough was learned, however, to indicate that many of the cases which are reported are the result of importation, due in large measure to the rather general custom of "visiting in the country," a custom not confined to the upper, but extending also to the lower classes. Otherwise, infection is received through the many channels associated with unsanitary surroundings.

Summary and Conclusions.—In view of the many factors which have exerted an influence upon the causation of typhoid in the District of Columbia and the rather complex relation which they bear to the continued prevalence of the disease during the year, it may be well to summarize the results of the foregoing facts and studies in a series of conclusions, as follows:

1.—The mortality from typhoid, as well as allied diseases classified as typho-malarial, malarial and diarrhœal, has decreased steadily, though in an irregular manner, from 1890 to the present time; the causes for which appear to be due to a number of factors. Of these the principal are the following:

(*a*) A purification of the water supply by subsidence in one or more of the storage reservoirs.

(*b*) A decrease in the number of public wells and box privies in the District.

(*c*) A closer supervision and control of the milk supply of the District.

(*d*) Other minor and more general improvements in the sanitation of dwellings, better medical care, etc., etc.

2.—During the present year, as shown by the records, the mortality and morbidity from typhoid have shown no considerable reduction, as compared with the last two years.

3.—From as reliable sources as could be obtained, there appears to be an unusual prevalence of typhoid in other portions of the country, outside of the District of Columbia, during the present year, which would indicate that the prevalence of typhoid within the District has been relatively less.

4.—The high bacterial efficiency of the new filtration plant and the low number of bacteria in the effluent are *prima facie* evidence

Mr. Horton. of the high degree of purity of the public water supply. This indicates clearly that the new filtered supply cannot be considered as not contributing in any appreciable degree to the continued prevalence of typhoid in the District, but, on the contrary, it has probably been the sole cause of keeping down the typhoid rate during the season when the disease has shown an unusual prevalence in other localities outside the District.

5.—The water from the Potomac River water-shed, having a resident population of not more than 40 persons per sq. mile, and purified by sedimentary storage basins to the extent of a 75 to 90% removal of bacteria, has probably never influenced, to the extent commonly supposed, the typhoid rate of the district, except at times when these reservoirs were not in use.

6.—The cause of the continued high mortality from typhoid, shown by statistics for the District, is due to a number of causes, some of which are complex, but each of which contributes its share to the total mortality. Among these may be mentioned the following:

(*a*) A relatively large proportion of imported cases contracted during the customary summer visits of a large number of all classes of inhabitants. Dr. Kober estimates the cases attributable to this infection to be from 15 to 20 per cent. In the writer's opinion, this is perhaps under-estimated.

(*b*) Occasional infection due to public wells, especially the shallow wells subject to frequent surface contamination.

(*c*) Serious infection from box privies, which admit of the spread of disease through the agency of flies, etc.

(*d*) Infection through the medium of the milk supply. Although this supply is subject to careful inspection, yet as handled in Washington, it is a continual source of infection; that outbreaks occur with periodic frequency is substantial proof of this. Dr. Kober estimates that milk is probably responsible for 15% of the cases of typhoid in the District.

(*e*) Infection through the medium of oysters. This cause undoubtedly has a larger share in the infection by typhoid than has been accepted previously. Recent investigations by H. W. Clark, Chemist, Massachusetts State Board of Health, have shown very clearly that the ordinary cooking of oysters and clams removes a relatively small percentage of the infection. Some oysters from sewage-polluted beds are eaten in large quantities, which accounts for at least a number of cases.

(*f*) Infection through the eating of unsterilized vegetables, fruit and berries. These are consumed in large quantities, especially in summer, and the sources of infection are many—starting with the fertilization of the soil with human manure, then the gathering by persons who are compelled to nurse patients in the country, and finally their exposure to infection from flies, in the markets. Mr. Horton.

(*g*) The agency of flies and other insects.

(*h*) A host of other means of infection of such a nature that the germs are transmitted from the intestines of one victim to those of another through routes which are so many and diversified as to defy detection.

7.—Although there is a strong presumptive evidence that the filtration plant has already exerted an influence in preventing a greater prevalence of typhoid during its operation this year, and that, as was the case with the filter plant at Lawrence, Mass., it will show in future years a more marked reduction in the typhoid rate, any hope for a permanent and further reduction of typhoid must be looked for in further improvements in house sanitation, the abolition of wells and privies, more effective oversight and control of the milk supply, more vigorous enforcement of disinfection during typhoid illness, and a reduction of the more varied and complex causes of infection inherent in the daily lives of the people, especially among the lower and more ignorant classes.

In closing, the writer wishes to acknowledge his appreciation of the courtesies extended by the following gentlemen, in placing at his disposal official records and other information indispensable to his studies: The District Health Officer, Dr. William C. Woodward, and his corps of assistants; General Walter Wyman and Dr. M. J. Rosenau, of the U. S. Public Health and Marine Hospital Service; Mr. M. O. Leighton, Chief of the Hydrographic Division of the U. S. Geological Survey; Lieutenant E. N. Dent and Messrs. E. D. Hardy and F. F. Longley, of the Filtration Bureau; J. S. Fulton, M. D., of Baltimore, Md.; George M. Kober, M. D., of Washington, D. C.; E. C. Levy, M. D., of Richmond, Va.; W. W. Wiley, of Cumberland, Md.; and W. R. Hall, Hydrographer, U. S. Geological Survey, of Atlanta, Ga.

JOHN H. GREGORY, ASSOC. M. AM. SOC. C. E. (by letter).—The writer has been particularly interested in that part of the paper relating to the transportation of sand by water and the cost of handling sand in the regular operation of the filters. In a previous discussion* on the filtration of water, the writer gave a brief historical review of sand washers of the ejector type in America, Mr. Gregory.

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 250.

Mr. Gregory. and of the development of the method of transporting sand by water as applied to filtration works, stating that, in his opinion, when an ejector is used in connection with a washer, two hoppers in the washer would be ample for washing the sand, and that satisfactory results could probably be obtained with one hopper. The writer would ask whether any attempt has been made to operate the washers with only one hopper, and, if so, what results were obtained?

The authors are to be congratulated on the remarkably low labor cost of handling sand. To indicate what a satisfactory showing has been made, it will suffice to compare the cost at Washington with that at Philadelphia in 1904 and 1905, and with that at Albany in 1899 and 1900, during the first year and a half of operation there, as given by Mr. Vogleson in Table 18.

For an intelligent comparison of these figures, it is essential to know the rate per hour paid for labor. At Albany during the period cited, the rate was $1.50 per day of 8 hours, or $0.18¾ per hour. Mr. Vogleson gives the rate at Philadelphia as $1.75 for 8 hours. The authors do not state the rate at Washington. The Philadelphia works are cited, as being most nearly comparable with those at Washington, the scraping, ejecting and washing being in general similar, the sand, however, being replaced by contract. At Albany the sand was wheeled out to the court and dumped, then rehandled through the washers, and later replaced in the filters by wheel-barrows. A marked economy is shown in the newer methods used at Washington over the older ones used at Albany.

Practice in the method of handling sand has rapidly changed since the Albany filters were built. At Philadelphia and Washington the sand is scraped into piles by hand, then shoveled into ejectors and transported by water to the washers located in the court. From the washers the sand is again transported by water to storage piles or bins, and, after a sufficient quantity of sand has accumulated, it is replaced in the filters.

At New Haven, Conn.,* no court has been provided, the sand from one filter being ejected by water to a washer and then transported by water to another filter held in reserve to receive it.

A similar method was recommended, by the Commission on Additional Water Supply of New York in 1903, for the filtration works proposed for the new supply. A similar design was also adopted at Pittsburg in 1904 for the works now under construction there, except that machines traveling above the surface of the filters will be installed for scraping and ejecting the sand, thus doing away with a large amount of hand labor.

Still other methods of washing and handling sand are receiving consideration at the present time, the most promising being those

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 254.

whereby the sand will be washed without removal from the filter, a feature greatly to be desired. Although the writer has spent considerable time in studying one of these methods, he does not feel at liberty at present to discuss this phase of the subject at length. There is one feature, however, to which he would call attention. In a mechanical filter, the sand is not removed, but is washed in place by an upward flow of water. At Little Falls and at New Milford, N. J., air is used to assist in agitating and washing the sand layer, and at the works now under construction in Columbus, Ohio, an air installation has been included. At one other large plant now being built, reliance will be placed solely on wash-water applied at a rate higher than that commonly used. If a mechanical filter can be washed satisfactorily with an upward flow of water, why cannot the same principle be applied to a slow sand filter? The writer realizes that with a slow sand filter, constructed so as to be washed with an upward flow of water, assisted if necessary by agitation of the upper surface of the sand layer, many interesting and difficult problems present themselves for study and solution, but it is his opinion that none of them is insurmountable. He has had this general design in mind for several years, and believes it is worthy of consideration by those who are interested or engaged in the design and construction of filtration works. Mr. Gregory.

ALLEN HAZEN AND E. D. HARDY, MEMBERS, AM. SOC. C. E. (by letter).—The general fact brought out in the discussions of Professor Sedgwick, Drs. Magruder, Kober, Woodward and others, that the filtration of the water supply at Washington has made no material change in the typhoid situation, is one of the most important sanitary developments of the time. The writers cannot express too highly their gratification at the thorough manner in which this subject has been investigated and discussed. They believe this to be the most important result of the paper, and they regard the discussion as being so complete and conclusive that it is not necessary for them to attempt to add to this part of it. Messrs. Hazen and Hardy.

Dr. Woodward mentions a number of points in regard to the construction and operation of the filters. It is true that at Washington the fine particles were removed from the sand to a somewhat greater extent than at Albany. It is not true, however, that the Washington sand had these particles removed to an unusual extent. The sand used in the large German plants has been washed fully as much as that used at Washington. The English sands have had their fine material removed to a much greater extent. The Washington sand, on the whole, is a little finer, rather than coarser, than the average, and contains somewhat more fine material. The writers can assure Dr. Woodward that no appreciable improvement in the action of the filters could be secured by using a less perfectly washed sand.

Messrs. Hazen and Hardy.

The suggestion that the operation would be helped by using the by-passes, cutting out the sedimentation reservoirs and reducing the period of sedimentation, the writers also believe to be impracticable and not tending to the best results. They believe that the failure to produce a clear effluent is not due to the removal of too much material in the sedimentation basins, but to the fact that too much fine material passes entirely through the sedimentation basins, and does this because it is so fine that it is incapable of being removed by them.

On the turbidity question, the writers also are inclined to take a somewhat different position from that expressed by Dr. Woodward. They cannot agree that the production of turbid effluent does not tend to increase the typhoid rate.

People get used to clear water very rapidly. When they are used to it they will not willingly take turbid water. When the water in the pipes is temporarily turbid, as it has often been during the past year, a great many people will turn to the hundred public wells, more or less, and the uncounted private wells, from which they can obtain clear water, and certainly many cases of typhoid will be caused in that way. These are indirectly the result of the turbid condition of the public supply.

The writers believe that a reasonable standard to be reached in this case is the production of water of such a degree of purity and attractiveness that it is equal to the best spring water. Such a water, regularly supplied, would leave no demand for spring water, nor for the use of shallow wells; and the acceptance of a lower standard is not in the interests of public health.

Dr. Woodward suggests that the responsibility for proving that coagulant is not injurious to health rests upon those who propose it. The writers are prepared to take the matter up upon this basis, and respectfully submit that the evidence that coagulant can be used without results injurious to the health of those using the water is as complete as can be reasonably expected.

Coagulant is now used systematically in treating the water supplies of more than 40 cities in the United States with more than 25 000 inhabitants each. In 1904 the population of cities using coagulant exceeded 3 000 000, and at the present time the number has increased to at least 4 000 000.

Coagulation has been practised systematically in public water supplies in the United States for the last twenty years. Its use to a limited extent was known and practised one hundred years or more ago.

In Europe, chemical treatment is used regularly in connection with sand filtration at Bremen, Germany, in times of flood; at Schiedam, Holland; and all the time at Antwerp, Belgium, and at many of the smaller Dutch cities, and this use is rapidly increasing.

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Coagulation is unnecessary and superfluous for some waters. These are the clear Northern waters from above the line of the glacial drift, etc. For the highly turbid waters of the South and West, including the Potomac, it is the only known method of treatment which is thoroughly effective.

Notwithstanding the extensive and rapidly increasing use of coagulant in the United States during the last twenty years, the writers have yet to hear of any case where it has even been claimed seriously that a municipal supply treated in this way has been injurious to the public health. If there were injurious results, they would certainly have been discovered before this by the health officers and doctors of the cities where the method has been used.

This is more certain because of the strong prejudice which has existed against the use of coagulation, and which Dr. Woodward's discussion makes clear is not quite dead. This prejudice has been strong enough to make many engineers, chemists and doctors extremely alert for any injury to health which might be attributed to it.

Injurious results have not been observed because they do not exist. And they do not exist, first, because the substances used in coagulation are not poisonous, nor even injurious unless used in vastly larger quantities than are necessary in water purification; and, second, because the alumina or iron is eliminated from the water in the course of the process, and does not appear in the effluent.

The use of sulphate of alumina does not increase the proportion of alumina in the effluent. On the other hand, it often decreases it, and no trace of the alumina applied can be detected in the effluent under conditions of filtration like those proposed in Washington. The Potomac water carries naturally a certain quantity of alumina. This is now reduced in the course of purification. It would be reduced still more with coagulant.

The other part of the sulphate of alumina, namely, the sulphuric acid, combines with the carbonates, always present in the Potomac water in much more than sufficient quantity to combine with the acid, and calcium sulphate is formed. For domestic purposes this substance is not more objectionable than the carbonate from which it is formed. For boiler uses it is slightly more objectionable. The change is so slight that practical difficulty has not often been experienced in boilers with waters treated in this way; and the theoretical objection on this ground, which is freely admitted, must be taken as a minor offset to the great advantage obtained from the treatment.

Mr. Maignen is disturbed by the fact that the dirty wash-water is sent to the bins with the sand and may contaminate it. With sand washers of the Albany type there might possibly be some ground

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for this criticism. With the washers used at Washington, the separation of the dirty water from the sand is more complete, and the sand is carried to the bins by comparatively clean water. Actually, the washed sand drawn from the bins is cleaner than the sand when originally put in the filters.

The writers believe that Mr. Maignen is mistaken in his computations as to compensating orifices. If he will go through the whole calculation, he will find that the sizes of the orifices are the same, whether the computation is made for a one-million rate or for a four-million rate, and the reasons for this are stated clearly in the paper.

Mr. Maignen criticises the arrangement of the regulation, especially the automatic reduction in the rate when there is not a demand for water, caused by the backing up of the water in the pure-water reservoir. The writers refer to the discussion of this matter in connection with the Albany paper.* They will add at this time that the method of regulation used at Washington is that in general use in Europe and in most of the filter plants in America.

Objection to the old procedure was first raised by certain authorities in Germany in 1893, and certain regulations were promulgated; but the water-works men did not accept the changes readily, and made such persistent efforts that the rule was shortly modified, and since then has not been enforced in Germany.

The advantages in the method of regulation used at Washington, in economy of construction and convenience of operation, are very great. The Philadelphia filters, and perhaps a few other plants, have been built on other principles, but the writers see no reason for wishing to modify the arrangement used at Washington.

Variations in the rate of filtration due to the backing up of water from the pure-water reservoir in the Washington plant are small and gradually applied. The nominal elevation of the water on the filters is 4 ft. above that ever reached by the water in the pure-water reservoir, while, as a general rule, the differences in elevations are considerably greater. Frequently, for periods of several days, the rate of filtration is not affected in the least. The maximum change in the rate from the highest to the lowest in 24 hours is about 500 000 gal., the highest rate for the day being the predetermined normal rate.

The effect of changing the rate of filtration has been studied at Washington with changes much greater and more suddenly applied than any which have ever been caused by the backing up of water, and no deterioration in bacterial efficiency has ever been found to occur; so that the objection to the process, for this plant at least, is purely academic, and does not rest upon actual results.

Transactions, Am. Soc. C. E., Vol. XLIII, p. 288.

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Mr. Maignen must be mistaken as to his understanding of the outlets. Certainly, no such effect upon the sand as he describes is produced.

The writers are particularly glad that the subject of preliminary filtration is brought up for discussion. They have followed this subject with much interest; and have studied the results obtained at Philadelphia, at Bethlehem, and at Lancaster, and experiments made at other places. As a result of this study, they state without hesitation that, in their opinion, no system of preliminary filtration like that used in the cities mentioned, or with any reasonable variations and improvements, would suffice in Washington to produce effluents of reasonable clearness; and, further, they believe, that no system of successive filtrations, within reasonable limits of rate and cost, would suffice to clarify Potomac water thoroughly. This is based on experience and tests at other places. The theoretical reason for it was discussed in a paper "On Sedimentation,"* and also by R. S. Weston, Assoc. M. Am. Soc. C. E.†

The writers understand that at Philadelphia the effluent from water filtered through preliminary filters and standard sand filters has occasionally been turbid; and at Lancaster, Mr. Maignen has been compelled to resort to the use of coagulant to maintain a clear effluent.

At Bethlehem, a naturally coagulated water is applied to the filters. This is a particularly interesting condition, and comes about in this way: The Lehigh River above Bethlehem receives a large quantity of mine drainage which contains sulphate of iron, and some miles above the town the water in the river is normally acid, and always contains iron in solution. Immediately above Bethlehem the river receives from limestone regions large tributaries which are strongly alkaline. The mingling of the waters results in the precipitation of the iron. This precipitation takes place on such a scale that it can usually be observed in the river when passing in a railway train. The water coming to the filters is thus thoroughly coagulated. When this natural coagulation is occasionally interfered with by floods, the intake is closed, and the supply is maintained from the two reservoirs, which are fortunately sufficient to keep up the supply for some days. The plant which Mr. Maignen designed takes full advantage of this situation, and he deserves every credit for having utilized it.

It is also to be noted that the Lehigh River is by no means as turbid as the Potomac. The operation of the filters in Bethlehem has been generally satisfactory, but there is no reason to believe that corresponding results could be obtained by the same process at Washington.

Transactions, Am. Soc. C. E., Vol. XLIII, p. 45.

†*Transactions*, Am. Soc. C. E., Vol. XLIII, p. 74.

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As preliminary filtration cannot be depended upon to produce a clear effluent, the only possible reason for using it is to reduce expense. By referring to the tables in the paper it appears that the items of cost which might possibly be reduced by the use of preliminary filters are the labor and supplies for cleaning filters, amounting to about 60 cents per million gallons. It will be observed that, in order to make a saving in total expense, the reduction in these items must be made to pay, first, the operating expenses of the preliminary filtration, including the cost of pumping to an additional height to overcome loss of head in the filters and in the piping connections, etc.; second, the capital charges on the additional installation required; and, third, there must be a saving of some magnitude to justify the additional cost and complexity of the plant. The writers believe that it is obvious to anyone familiar with water purification that, after making the allowances above mentioned, no such saving can be made.

Mr. William B. Fuller must have been misinformed as to the relative elevation of the filters and the pure-water reservoir. These are not at the same elevation, and the water can never rise in the pure-water reservoir to the same height as that in the filters. The elevation of the springing line in the pure-water reservoir is 3 ft. below that of the filters, and, as explained in reply to statements made by Mr. Maignen, the water actually never rises within 4 ft. of that in the filters. The backing up of water, therefore, affects only slightly the rate of filtration, and does not, to any material extent, tend to cause the entire plant to go out of service at the same time.

The hose now being used for transporting sand is heavy, 3-in., double-jacketed, fire-hose, with a rubber lining. It was purchased from the Fabric Fire Hose Company, of New York, and, under the severe usage it receives, it seems probable that its life will be one year.

The writers agree with Mr. William B. Fuller to a certain extent that the replacing of washed sand directly in a filter has certain advantages over storing it in sand bins. This method allows for more artistic treatment, and would give a better general appearance to the surface above the filters. For large plants, however, the distances the sand would have to be carried would probably be so great that the plant would have to be divided into several groups, and one filter in each group would have to be kept out of service to receive the sand from the other filters in that group. As a consequence, it is probable that more filters would be kept out of service than by the plan at Washington, and that the saving in area by reducing the courts would be more than balanced by the increased number of filters out of service.

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Mr. Vogleson's description of the experiments and results at Philadelphia is particularly interesting. While the experiments upon the friction of mixtures of sand and water in pipes recorded by him differ considerably from those shown by the plottings used for the purpose of design at Washington, the discrepancies are sometimes in one direction and sometimes in the other; and, on the whole, the agreement is sufficiently close to support the accuracy of the general method of computation, and affords a valuable corroboration of the general correctness of the results.

The proposition advanced, that under certain conditions a 3-in. pipe is capable of carrying more sand than a 4-in. pipe, is of course correct. The condition under which this may occur is that with small quantities the velocity produced in the 4-in. pipe is below that required for steady and regular flow. On the other hand, there cannot be the slightest doubt that an ejector adapted to service with a 4-in. pipe would throw twice as much sand through a long 4-in. pipe as through a long 3-in. pipe. The fact that this condition was not obtained by Mr. Vogleson simply indicates that the ejector actually used was not adapted to the 4-in. pipe, and this experience does not indicate that 3-in. pipe under any circumstances is equivalent in carrying capacity to 4-in. pipe. The apparatus for feeding the 3-in. pipe is somewhat lighter than that for feeding the 4-in. pipe, and in small plants there may be reason for using it; but, for a large plant, the writers, as a result of their experience, are disposed to take the view that 4-in. pipe, worked to its proper capacity, is a more economical size than 3-in. pipe.

Mr. Henny may be interested to know that ivy has been planted with the intention of covering the bare concrete spaces to which he objects. The whole filter plant has been made into a park, named "McMillan Park," in honor of Senator McMillan, who was largely instrumental in securing the establishment of the plant; and it is hoped that the treatment of the whole area can be made in a satisfactory way, and that it will add substantially to the attractiveness of this part of the city.

The French system of constructing reservoirs, described by Mr. Labelle, is very interesting, and certainly results in a large reduction in the quantity of masonry; but this saving, under European conditions, with expensive materials and cheap labor, is larger than under American conditions. This is particularly true where concrete is the material of construction, and where the cost of the forms is not reduced with the reduction in the masonry, but, on the other hand, is often increased. Nevertheless, if the time allowed for the preparation of the plans had permitted, the writers regard it as certain that a somewhat more economical design for the outside walls could have been secured, as was indicated in the paper.

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Miss Blatch's discussion adds greatly to the available data upon the flow of mixtures of sand and water. The writers are pleased to note that the method of computation which they used in making the design, though based on only a limited number of experiments, is in general accord with the other results which Miss Blatch mentions.

The discrepancy which she finds in the writers' diagrams is below the point of economical use. It is interesting, but not of great practical importance.

Mr. Gregory has given us a glimpse into the future of sand handling and washing which is very interesting. There will doubtless be many changes and improvements in this feature of filters in the near future. These developments will be watched with great interest by engineers, with special reference to the relative cost of operation for plants where the various systems have been installed.

In answer to his query regarding the number of hoppers necessary to wash the sand satisfactorily, the writers can assure him that, with the apparatus used at Washington, one hopper is found to be sufficient. Two hoppers were provided, and both were kept in service up to November 1st, when the piping was changed; since that time only one has been used.

The rate of pay for laborers at Washington is $1.50 per day of 8 hours, and, in computing the cost for sand handling, that for supervision is included with the labor charges.

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TRANSACTIONS.

Paper No. 1037.

STEAM LOCOMOTIVE AND ELECTRIC OPERATION FOR TRUNK-LINE TRAFFIC— A COMPARISON OF COSTS AND EARNINGS.*

By Joseph Mayer, M. Am. Soc. C. E.

With Discussion by Messrs. Frank J. Sprague and Joseph Mayer.

The only adequate motive that will lead to the change from steam to electric operation is a large return on the investment required. The return on this investment is the increase in net earnings which is the result of the change. For finding this increase, the probable future earnings, if steam-locomotive operation is continued, must be compared with these earnings if electric operation is introduced. The comparison to be made is not one of present net earnings from steam locomotive operations with future net earnings from electric operation; both terms of the comparison are future earnings. On all lines with heavy local passenger traffic, where steam locomotive operation is maintained, parallel interurban electric roads will be built which will take from the steam roads two-thirds or more of the local passenger traffic. This has already happened on many roads by the competition of the expensive and slow direct-current trolley roads. The severity of this competition

* Presented at the meeting of November 21st, 1906.

will be greatly increased by the construction of much cheaper single-phase roads, which permit much higher speeds and larger cars and trains, without the use of the expensive and troublesome third-rail and rotary-converter sub-stations.

Greatly reduced passenger earnings with future steam-locomotive operation, therefore, must be compared with the probable earnings accompanying electric operation.

Experience on interurban electric roads has shown that small trains or single cars can be run cheaply with many stops, and at higher speeds than is financially practicable with steam locomotives, and that a large increase of local passenger earnings is the result. If electric operation is introduced on the present steam roads, it must, to gain the fullest possible advantage, be generally accompanied by a great change in the number and size of trains, their speed, and the number of stops. The cars for strictly local passenger traffic must, like the present interurban electric cars, wherever practicable, run through the streets of the urban centers of population, so as to give continuous service without change of cars, and to take the passengers, as nearly as possible, from their starting point to their destination. Fares of about 1½ cents per mile have been found by experience most profitable for the kind of service given by interurban roads. They will probably be most profitable where similar service is attempted on the present steam roads.

A surprisingly large amount of local passenger earnings per head of accessible population has been secured by interurban electric roads, and has never been approached by steam roads. The present local passenger service of the steam roads, in view of the results obtained by interurban electric roads, is entirely antiquated and utterly inefficient. Dwindling local passenger earnings are the inevitable result of persistence in present methods. The cost of electric operation which must be considered, therefore, is that for the new trains or cars, at the new speeds, with the new number of stops; and not that of electrically operating trains of the number and size, and with the speeds and stops which have proved most economical with steam locomotives. On all roads with large local passenger traffic, the increase in passenger earnings which will be secured by the changes in method of operation taught by the experience of interurban electric roads is the most important

gain. The increase of local passenger traffic will greatly reduce all costs per passenger-mile, it will increase the total investment required but little, and will thereby reduce the interest charges per passenger-mile, as compared with those with a smaller traffic. The profit per passenger-mile needed to make the new investment a promising one is reduced nearly in the same ratio as that of the increase of traffic.

With steam locomotives, the heaviest through passenger trains cannot be run at the highest present passenger-train speeds. The capacity of the boiler that can be fired by one man and accommodated on one locomotive is limited, and has been closely approached in the largest present engines. Several steam locomotives cannot be controlled by one man at the head of the train. Fast trains with several steam locomotives, therefore, are not practicable. With electric operation, the heaviest passenger trains can be run at the highest speeds, and several locomotives may be used safely and economically on fast trains by means of multiple-unit control. The heavy, partially balanced, reciprocating parts of steam locomotives, which, at highest speeds, cause violent lateral swaying and vertical hammer blows on the track, and thereby greatly increase the cost of maintenance of way, are absent in electric locomotives. At the highest speeds the economy of steam locomotives diminishes rapidly with increase of speed, that of suitable electric locomotives remains approximately constant. With electric operation, higher speeds, therefore, will probably be adopted for all through trains, and the length of the fastest trains can be increased when desirable. As regards the probable future amount of local passenger traffic with the new kind of service, the experience gained on interurban electric roads must be the guide. In view of these facts, a mere comparison of the cost of operating the present passenger trains with steam or electricity has only a remote bearing on the question whether a change to electric operation is advisable, and must lead to an entirely erroneous estimate of the net return from the required investment. The present size and number of trains, and their speeds and number of stops, are closely adapted to the capacity of the steam locomotive. Wherever this train service for local passenger traffic has come into competition with interurban electric roads, its inferiority, as regards capacity to secure traffic,

has been clearly demonstrated. It will be abandoned, therefore, at the same time as the steam locomotive. The best possible electric service must be compared with the best steam-locomotive service, to obtain any results of value. The investigation will naturally begin with a consideration of the conditions which govern the probable increase of local passenger traffic and earnings due to improved service, since this is known to be the most powerful factor in favor of the change to electric operation. The accessible present and probable future population along the line furnish the future earnings.

Commuters using the line twice a day give the largest earnings per accessible inhabitant. They are important mainly near large cities, where their number along a line may increase rapidly by improved service. In such situations no further argument is needed to prove the superiority of electric operation, and the transformation of such lines is going on at a rapid rate.

The opinion is more divided in regard to long-distance lines. If the population along such a line is considerable, exceeding 500 inhabitants per mile of line, within 4 miles on each side, there exists either a competing interurban parallel trolley line or one may be safely counted upon in the near future. The existing line is probably a direct-current line on which small cars run at moderate speed. This can, and probably will, be changed to a single-phase line on which large cars can be run at high speed. If it is favorably located, it will receive the bulk of the previous local passenger traffic of the parallel steam road and all the new traffic developed by the improved service. By a change to electric operation of the steam railroad before the parallel trolley road is built, the latter is made impossible unless the available traffic is very large or the steam railroad is unfavorably situated to secure it. If the competing electric road is already in the field, a fraction of the total local passenger traffic, largely depending in amount on the location of the two competitors and the quality of their local passenger service, will be obtained by the steam road by the change to electric operation. Statistics of earnings of interurban electric roads have shown that they amount to from $3 to $10 per inhabitant within 4 miles of the line, not counting those in a terminal city of more than 75 000 inhabitants. It is evident that the distribution of the

population along the line, whether it is mainly within 1 mile of the line or further distant, and whether it is in a terminal or at intermediate points, is important. The degree of prosperity of the population and the quality of the roads will greatly influence the amount of travel per inhabitant. The location of competing lines will often limit the accessible area. The present local passenger traffic on the steam road, without a present electric competitor, gives a further clue, together with the experience in regard to the amount of traffic which has been developed by interurban electric lines along steam roads of the same or different amount of local passenger traffic. Equal improvement in local service will probably produce a nearly equal percentage of increase in local traffic. A detailed study of all the pertinent local conditions along the line is required to ascertain the probable amount of local passenger traffic which will be secured by any proposed electric service on the existing steam road.

The increase in through passenger traffic due to the use of higher speeds and the greater comfort and cleanliness possible with electric operation will, in many cases, be large, especially until electric operation is adopted by the competitors. There will be a great loss in the through traffic of the steam road if the competitors are first in the change to electric operation. The percentage of increase in through traffic cannot be calculated, but a moderate estimate will be nearer the truth than entire neglect of this gain.

The possible length of slow-speed freight trains using one engine is now mainly governed by the largest permissible draw-bar pull, which, with ordinary freight cars, is about 50 000 lb. With electric operation and multiple-unit control, it will be practicable to operate from the head several locomotives distributed through the train. Long trains can thereby be used on heavy grade lines, and the traffic capacity of two tracks can be indefinitely increased. On lines of large traffic, the possibility of operating passenger and freight trains of any desired length, and thereby reducing train expenses per unit of traffic and increasing the traffic capacity of the road, becomes very important. With high voltage in the contact conductor this is possible without much increase in the cost of the transmission line. Since accuracy in the estimate of the value of these advantages of electric operation is not possible, a

conservative estimate, which is probably below the truth, must be made, and, to justify a change, it must be shown conclusively that the necessary investment will bring a large percentage of return.

Next in importance to the increase of passenger traffic which may be secured by the adoption of electric operation is the reduction in the cost of motive power. This reduction is large where coal is dear, and especially so where a cheap water-power is also available within reasonable distance. At Niagara Falls power can be secured in large quantities with the load factor which will be obtainable in the electric operation of long lines of present steam railways for from $\frac{1}{4}$ to $\frac{1}{3}$ cent per kw-hr. If power is produced in large central stations along the line, by means of gas engines and cheap, inferior coal, it will cost about twice as much at a central station. In a modern steam-turbine station, under favorable circumstances, power with the load factor of about five-eighths, which is easily obtainable on a busy double-track line of about 50 miles which would be served by such a station, can be produced for 0.7 cent per kw-hr., inclusive of all charges for interest and depreciation of plant. The transmission to the point of consumption will be cheaper with power-stations distributed along the line than with one water-power station, as the distances will be less. Within 200 miles of Niagara Falls, and other equally cheap water-powers, there will be a material saving by the use of water-power. Electric operation, therefore, is at present most seriously considered near large cities where local passenger traffic is important, also where coal is dear, and where cheap water-powers are available. As regards the freight traffic, the problem appears to be much simpler, because the amount of traffic and the number and size of trains may possibly remain the same. Interurban electric roads, however, have begun to encroach on the freight traffic of the competing steam roads. As single-phase roads will be better suited for the running of trains than the present direct-current trolley roads, this competition will become more severe in the near future; it has been of such small importance in the past, however, that its future amount cannot be quantitatively estimated with the desired degree of accuracy. It is neglected, therefore, in the following calculations. On lines with large passenger traffic the freight movement is mainly carried on during hours of moderate passenger movement. This

fact greatly improves the load factor or the ratio between the average and the maximum amount of power used. If power is obtained from a water-power plant with practically an unlimited amount of water, its cost of production mainly depends on the maximum amount consumed at one time. Any power used at other times costs practically nothing. The cost of transmission is also governed by the maximum consumption. The addition to the cost of power and its transmission, due to the electric operation of the freight traffic, therefore, is small wherever the maximum amount of power needed is mainly determined by the passenger traffic. There is, therefore, a great difference between the cost of electrically operating a given freight traffic, considered as a separate problem, and the cost of electrically operating this same freight traffic on a line with a large amount of electrically-operated passenger traffic; the difference is greatest where water-power is used, but is important even with steam-power. The two traffics, freight and passenger, must therefore be considered together; and the cost of power and its transmission to the motors must be charged mainly to the traffic which can best bear it. It will be found profitable to operate either both by steam or both by electricity. If a large reduction in the cost of operating the freight traffic is secured by adopting electric operation, lower freight rates may be charged, where necessary, to secure the traffic. A great increase in freight traffic may thereby be the result of electric operation, making the lower rates the more profitable. The amount of future traffic, therefore, will depend indirectly on the new cost of operation. To determine the former, an approximate estimate of the latter must, therefore, be made. This leads to a consideration of the technical features of the problem. These features concern the electric motor cars and locomotives, the transmission system, and the central power-stations. These must all be adapted to the future amount and kind of traffic.

Voltage and Kind of Motors.

It is now generally admitted by the most eminent electrical engineers that the use of direct currents, at 600 to 700 volts, for transmitting the power to the locomotives by means of a third-rail is uneconomical for the heaviest traffic. Direct currents at 1 500 to 2 000 volts, and alternating currents of 10 000 to 20 000 volts, and a

suspended contact conductor, are advocated for this purpose. The Zossen experiments, near Berlin, Germany, with high-speed trains, confirm the conviction of a few years ago of the leading electrical engineers of Germany. They may be summed up as high-speed, and high-voltage alternating currents. At that time three-phase currents with induction motors were adopted. The single-phase motor has since been developed. The investigations which are now under way, in Sweden, in Switzerland, and in the United States and other countries, for ascertaining the best system of electrically operating the present steam roads, all assume as proven the necessity of higher voltage in the contact conductor from which the current is gathered by the trolley wheel or a sliding collector. The demonstrable savings in the transmission system, due to the use of high voltage for heavy traffic, are so large that they leave no doubt in regard to its adoption. The proof by experience that high-voltage alternating currents can be safely used in the trolley wire, the invention of suitable alternating-current motors, and the great reduction in the cost of electrically operating the heaviest traffic which is thereby obtainable, promise to make the electric operation of trunk lines under favorable circumstances very profitable. A discussion of this subject a few years ago—before these events—had convinced the most competent engineers that the electric operation of trunk lines, with enlarged street-railway apparatus and the then usual voltages, is not profitable.

For the problem under consideration, the most important transformation of an ordinary steam railway with freight and passenger traffic into an electric road is that of the Valtellina Railway, a branch of the Rete Adriatica. This branch, near Lake Como in North Italy, is 66 miles long; it has a large passenger traffic, a convenient water-power, and is in a country of high coal prices, all factors very favorable to electric operation. It has been in successful operation since September, 1902. Three-phase current, at 20 000 volts, is produced directly by the dynamos in the water-power central station situated on the line. This is sent to transformer stations on the line where it is reduced to 3 000 volts. This is carried by two suspended trolley wires and the rails to the locomotives, where it is used by three-phase induction motors, which can be connected either singly or two in cascade for two economical

speeds. Regulation for all other speeds is obtained by water rheostats. The main objections to this system are the double trolley wire, which makes very high voltage difficult to handle, and the poor economy at all speeds except two. The double trolley wire can be avoided by the use of single-phase current, and good speed regulation can be secured by the use of variable voltage obtained from transformers with a variable ratio of transformation.

A suitable single-phase motor was unknown a few years ago. Several such motors are now available. The motors extensively used on interurban trolley lines are: in Europe, the Winter Eichberg motor, of the Allegemeine Elektricitäts-Gesellschaft, of Germany; and, in America, the Lamme motor of the Westinghouse Company. Neither of these motors has yet been made in sizes large enough for building four-motor locomotives for the heaviest present trains. Such motors should have a capacity of about 500 h. p., and should be able to carry 50% overload for a limited time. Their size must be such that they can be placed on a locomotive of moderate, rigid, wheel base. The largest motors thus far constructed by the Westinghouse Company are of 250 h. p., and two locomotives, each with four motors, are required for the heaviest freight trains or the fastest through passenger trains. These, however, can be operated by one man by means of multiple-unit control, and the main objections to the use of several locomotives on one train are thereby removed. These locomotives use high-voltage alternate currents, and have excellent speed regulation, giving practically uniform efficiency at any considerable speed, two advantages which far outbalance, especially for trunk-line traffic, their inferiority in other respects. This latter may be summed up as great weight per unit of power, limited capacity, high cost, and only moderate efficiency at any speed.

By the use of high-voltage alternating current on the locomotive the cost of the current transmission plant is much reduced, and the power losses in it are greatly diminished; the converter substations, with their large power losses, are either entirely saved or replaced by a far smaller number of much cheaper transformer stations with small power loss. The total result, for trunk-line traffic, is a great reduction in cost of construction, maintenance and operation, and some saving in the amount of power needed at

the power-station. The only commercial experience with alternating current operation for moderately heavy traffic is with the three-phase locomotives of Ganz and Company on the Valtellina Railway. Their system has recently been adopted for the operation of the Simplon Tunnel. At present, the preponderance of theoretical argument is strongly in favor of single-phase currents for the operation of roads of varied and heavy traffic. The Swedish Government Commission, investigating the subject of electric traction for their steam railways, assumes this question as settled, and limits its investigation to various single-phase motors. In the experiments in Switzerland by the Oerlikon Company, single-phase motors are also used.

Westinghouse single-phase locomotives have recently been adopted by the New York, New Haven and Hartford Railroad for the four-track line from New York to Stamford, and by the Grand Trunk Railway for the Sarnia Tunnel and approaches. It is evident that the tests with these locomotives have convinced the responsible officers of these companies that suitable single-phase locomotives can be constructed. For the steam railway from London to Brighton, England, single-phase operation has recently been decided upon. For the Pennsylvania line from Camden to Atlantic City, with a similar kind of traffic, direct-current operation has been adopted. So much practical experience has been obtained with direct-current operation with the kind of traffic on these two roads, which consists of frequent small trains, that the results which can be achieved by it are accurately known. The adoption of single-phase operation for the London-Brighton line is clear proof that its theoretical advantages, even in a field which has in the past been considered especially that for direct-current operation, must be considerable, because, in the opinion of the responsible officers, they succeeded in overcoming the advantage of extensive past experience with direct-current operation of similar traffic.

Schuler and Steinmetz have developed the repulsion-induction motor for single-phase alternating currents, but no tests with such motors of large sizes or locomotives containing them have been made, as far as known to the writer. Another method of using the power received from the contact conductor, in the shape of high-voltage single-phase current, is the Ward Leonard system.

The current is transformed to lower voltage, then, by means of an induction motor and a direct-current generator, into direct current, which is used in direct-current motors. This system is not suitable for single cars, but can be used on electric locomotives, and gives excellent and economical speed regulation. Its complication, moderate economy, inevitably high cost, and great weight, are its serious drawbacks.

All the single-phase motors mentioned have a commutator, and can also be operated with direct current. This last feature is valuable during the present transition. There is no commercial experience with single-phase locomotives able to handle trunk-line trains. No convincing proof can be furnished in regard to their exact cost of maintenance. There are, however, good reasons to believe that it will be less than that for steam locomotives of equal power.

At present, single-phase locomotives promise the simplest and cheapest method of operating large trunk-line trains. The following estimates, therefore, assume their use. For certain kinds of traffic, direct-current motors with much higher voltage than that used at present will probably be used extensively in the future, but they are not considered in the following. Three-phase induction motors may also continue to be used, but will not be considered here.

Power Transmission.

The motors receive the current through a transmission system from one or more central power-stations. The transmission system, where the source is a water-power central station feeding a long line, will consist of the high-tension lines; the transformers, located in stations along the line, for reducing the pressure; and the contact lines on the side of, or over, the center of the tracks, either with or without feeders. Where heat power-stations are used, they will be at favorable points along the line, where water and coal are cheap, at intervals varying with the amount and kind of traffic and type of machinery used. Where favorable sites for power-stations can be found at distances of from 40 to 60 miles, 20 000-volt, single-phase current can be produced directly at the dynamos and sent to the locomotives without transformation; or lower voltage and shorter distances may be used, especially where the traffic is large and con-

sists of numerous units of moderate size. The use of the highest practicable voltage is especially desirable with very large but infrequent trains.

As a very high and dangerous voltage must be used in the suspended conductor, it is essential to suspend it in such manner that it will never come down. Trolley wires sometimes break and fall. Their method of suspension, therefore, is acknowledged to be inadequate. To prevent their falling, they are suspended from either one rope or wire immediately above, or from two ropes or wires in inclined planes. These carriers are of steel, and have considerable deflection between supports. Where only one rope is used for supporting a 000 or 0000 grooved copper wire, the strongest winds normal to the line will deflect to the leeward the originally vertical plane, containing the carrier, the suspenders and the trolley wire, so that it forms an angle of about 45° with the vertical. In this position, an ordinary trolley wheel leaving the wire while running at high speed would often strike the suspenders connecting the trolley wire and carrier. This would break the suspenders and give a violent jerk to the wire. It might thus break it and cause its fall. The ordinary trolley wheel, therefore, is not safe at very high speeds. A rolling contact, however, is desirable in order to diminish the wear on the trolley wire and on the collector. On the Valtellina Railway, with maximum speeds of about 50 miles per hour, long brass rollers with roller bearings have proved most satisfactory. This line has two trolley wires not suspended from ropes. On other roads sliding bows are used successfully. It is important that the point of contact should travel along the bow, as there will be rapid wear if this is not the case. To secure this, the trolley wire is not quite parallel with the track, but runs alternately to the right and left. With the high speeds of trunk-line trains and the very high voltages in the trolley wire, the greatest possible caution appears to be desirable.

To give more rigidity to the trolley wire, it is put under tension. This tension, however, varies greatly with changes of temperature. The suspenders are made stiff to prevent the rising of the wire with the passing of the sliding bow or roller. With a high voltage, the insulators are expensive and the leakage is considerable. Where a railway has great traffic, and can obtain money

at 4%, a substantial structure having small cost of maintenance becomes more advantageous than a short-lived, cheap, wooden structure. The timber poles of interurban trolley lines, therefore, are avoided, and the carrying ropes for the trolley wires are supported on steel bridges spanning the tracks, with steel towers at each side. With such bridges, long spans are more economical than short ones. With long spans, the vertical sag of the rope carrying the trolley wire must be considerable at the highest temperature. The wind will then blow the trolley wire out of reach of a sliding bow of moderate length. In this case, therefore, a trolley wire carried by a single rope is not satisfactory. It must be carried by two ropes in inclined planes meeting at the wire.

A satisfactory contact conductor which will be safe at the highest train speeds is one of the main problems to be solved before electric trunk-line operation can become a complete success.

The method thus far followed, in designing a new trolley wire suspension, has been empirical. The size of the wire and the number of supports have been increased, but no attempt has been made to calculate the effect of these changes on the strains produced in the contact wire by the sliding bow and the changes of temperature. A theoretical solution of this problem has been attempted in "The Mechanical Theory of the Contact Conductor for High-Speed Trains," which, to avoid interruption of the main argument, is introduced as an Appendix to this paper. It is evident that the subject under discussion cannot be treated exhaustively within the limits of a paper.

Amount and Kind of Traffic.

What is here attempted is to show by means of a single instance, which is typical of a great many others, what financial results may be expected from the change from steam to electric operation of trunk lines. The instance used is the Lake Shore Railroad from Buffalo to Cleveland. The writer's information in regard to present conditions is derived from the published report to the stockholders for the year 1905. This report refers to the whole system. The cost of steam-locomotive operation of the assumed amount of traffic on the selected part of the line can be inferred from the published data with a fair degree of accuracy. For the purpose of this argu-

ment, an approximate estimate of the present amount of passenger and freight traffic is sufficient.

The number of passenger trains at present is given in the time-tables. There is in existence a competing parallel electric road which would undoubtedly be improved if steam-locomotive operation were maintained. The local passenger traffic, which has already suffered severely from this competition, would decrease still further. There are now approximately 28 daily passenger trains going over the line, 4 being local trains. The probable future traffic with steam-locomotive operation is estimated at 26 daily trains for through passengers and 2 accommodation trains for local passengers. The average number of cars per passenger train is estimated at 8, and the earnings per passenger-train car-mile are estimated at 24 cents. This gives, for steam-locomotive operation, passenger-train earnings of $3 590 000. This includes $3 333 000 from through traffic and $257 000 from local traffic.

Increase in Passenger Traffic and Earnings.

The tracks are very crowded with traffic, and it is believed that it would not be expedient to run single cars or cars with trailers. The traffic for the shortest distances will probably generally remain with the present competing line. By establishing frequent fast local trains, the passenger traffic for medium distances can be secured. The combined population of the two terminals is about 900 000, that along the line is about 180 000. It is estimated that, by the service hereafter described, the local passenger earnings can be increased by $540 000, or $3 per head of population along the line. The increase in through-passenger-train earnings is estimated at 20% of the present earnings. The future loss in local passenger traffic by improved service on the competing electric road, if steam-locomotive operation is maintained on the Lake Shore, is estimated at $123 000.

The total difference in future passenger-train earnings between electric and steam-locomotive operation, therefore, is 20% of 3 333 000, or	$667 000
Increase in local passenger earnings	540 000
Loss in local passenger earnings avoided	123 000
	$1 330 000

Recently, it has been the policy of steam railway companies to buy the competing parallel electric roads. By such purchase and the consequent removal of competition for the local traffic, passenger fares can be maintained, which give a larger profit from the local passenger traffic of the two lines than would be obtainable by them when competing with each other and each lowering the fares for the purpose of securing a larger proportion of the total traffic. This increase in profit can be divided among the two companies, and the change of ownership can thereby be made profitable to both.

If the steam railway is electrified before the purchase of the competing electric line the prospective earnings of the latter are much reduced. This will correspondingly reduce the purchase price. The prospective loss of earnings due to the inferiority of steam railway operation can only be avoided by the introduction of electric operation, because when the steam railway purchases the competing electric road, the price includes the capitalized value of this loss of earnings. The avoidance of this loss, therefore, is one of the gains secured by the electrifying of the steam road even if the latter afterward buys the competing electric road.

Present and Future Trains.

At present there are, approximately, 28 passenger trains of eight cars, or 224 passenger-train cars, going over the line per day. These could not be materially reduced in number, in spite of decreased local traffic. Of these cars, 208 are in through trains. With 20% increase in through passenger-train earnings, 250 cars would be required for the through trains. For the $540 000 increase in local passenger earnings, with earnings of 24 cents per car-mile, 34 cars per day going over the line would be required. This gives 50 cars per day for local traffic, or a total of 300 cars per day for the whole passenger-train traffic.

The future traffic can be provided for by an average of 22 daily trains of 9 cars and an average of 21 daily trains of 5 cars, giving a total of 303 cars going daily over the whole line. In reality, the number of trains and of cars per train would vary with the seasons and, on different parts of the line, with the density of population along the line. As the increase in passenger traffic will be greatly affected by the schedule speeds, and as high-speeds can be

secured at moderate cost by electric operation with a cheap water-power, it will be advantageous to increase greatly the average train speed. An average increase of one-third over present speeds has been assumed in the following calculations. The through trains of 9 cars, therefore, should have a schedule speed of from 50 to 60 miles per hour, according to the number of stops. The 5-car trains should have a speed of from 30 to 45 miles per hour with stops from 2 to 6 miles apart.

Capacity and Cost of Electric Motive Power.

The 9-car train would weigh, with locomotives, 500 tons. The 5-car train, with 2 motor cars, would weigh 250 tons.

Two 1 100-h. p. electric locomotives would be required for the long trains, and three 500-h. p. motor cars would be needed for the short ones. The horse-power here given is not the commercial rating, but the average capacity of the motors in service.

The present freight traffic is estimated at 34 revenue and 3 work trains per day going over the whole line. It is assumed that the amount of freight traffic will not be affected by the method of operation. The average weight of the freight trains is estimated at 1 600 tons. The trains going east are much heavier than those going west. Two 1 100-h. p. locomotives would be needed for the heaviest trains. Two such locomotives can haul a train of 2 200 tons at 36 miles an hour over a level, straight track. If an average of two locomotives is used per train, much higher speeds can be maintained than with the present steam locomotives. Assuming 35 000 revenue train-miles per year per electric passenger locomotive, two-thirds of this number per electric motor car, and 25 000 per electric freight locomotive, which allows for the fact that each train is assumed to have two or three motor units, there will be required, for the above traffic, 42 passenger locomotives, 90 freight locomotives and 60 motor cars.

From various data, the following safe estimate of the cost of this motive power equipment is obtained:

42 passenger locomotives, each 1 100 h. p., at $33 000	$1 386 000
90 freight locomotives, each 1 000 h. p., at $30 000	2 700 000
60 motor cars, each of 500 h. p., at $18 000	1 080 000
Total	$5 166 000

Passenger Cars and Locomotives Saved.

The present passenger traffic requires 14 960 000 car-miles, or, with 45 000 car-miles per car, 332 cars. The future traffic will give 70 000 car-miles per car. It will require 76 032 000 car-miles per year, not counting the motor cars, of which there are 3 in each 5-car train. There will be needed 229 passenger cars; and 103 passenger cars will be saved. From data in the published report of the Lake Shore Railroad it is inferred that 150 locomotives will be saved.

The value of the rolling stock saved is estimated at $2 286 000
The net cost of new rolling stock, therefore, is.....$5 166 000
Less..... 2 286 000

$2 880 000

Amount and Cost of New Track Required.

The length of track at present on the 183 miles of line between Buffalo and Cleveland is estimated at 550 miles. The larger passenger traffic will require additional track. The necessary new track is estimated at 50 miles, its cost at $1 000 000.

For this traffic there would be required seven transformer stations having a total capacity of 45 000 kw., costing, complete, $8 per kw., or $360 000. For each track is needed a 1¼-in. suspended contact rod with suspenders 12 ft. apart; for each rod, one 1-in. carrying rope of 40 tons ultimate strength, with 7 ft. maximum vertical sag and 350-ft. spans. For every 350 ft. of line one overhead bridge will span the tracks and be supported between two steel transmission towers. The towers are required to carry the high-tension lines and feeders. To reduce to 4% the average loss in the secondary circuit, with maximum traffic and with 15 000-volt, 25-cycle, single-phase current, two No. 0, B. & S. gauge, copper feeders are required for each pair of tracks. The contact rods should be in insulated sections 2 miles long, the ends and center of the sections connected to the feeders by circuit breakers, so that any section of the rods can be cut out in case of a short-circuit. The rails must be bonded and cross-connected to serve as a return circuit. On account of the low conductivity of the rods for

alternating currents, the transformer stations must be nearer together than if 0000, B. & S. gauge, contact wires are used with the same relay feeders. The contact rods carry for most of the distance but a small current, and the rail return circuit to the transformer stations is shorter than with contact wires. The secondary circuit with contact rods, therefore, has about the same self-induction as the longer secondary circuit with contact wire.

The cost of the secondary circuit, the towers along the line for the primary circuit, and the transformer stations, all per mile of track, will be approximately as follows:

Supporting steel structure, insulators, circuit-breakers, etc.	$2 428
Transformer stations	600
Contact rods and suspenders	712
Ropes	608
No. 0 feeders and accessories	372
Track bonds, etc.	80
Total per mile of track	$4 800

This makes the total cost, for 600 miles of track, $2 880 000.

The cost of steel erected is assumed to be 3 cents per lb.; that of concrete foundations, $8 per cu. yd.

Power Transmission Towers.

Power would be obtained from Niagara Falls. Two lines of steel towers, each 25 miles long, would be required between Buffalo and Niagara Falls, at $2 000 per mile, and would cost $100 000. The cost of towers along the line is included in the cost of the contact line.

Amount and Cost of Electric Power.

It will be shown in the following that a maximum of 42 150 h. p. will be needed at the primaries of the transformers in the transformer stations along the line. The average distance of transmission to these stations is 117 miles. If the average voltage received is 50 000, and the loss in transmission 15% of the power leaving the power-station, if the power factor is 85%, and if single-phase transmission is used, then the quantity of copper required for

the high-tension transmission line is approximately 5 100 000 lb. This copper costs, in place, with accessories, about 22 cents per lb. This gives for its total cost $1 122 000. An aluminum line would probably be cheaper. The amount of power needed for the trains given above is obtained as follows:

The train resistance for freight trains is estimated at 8 lb. per ton. This is intended to cover the small amount of power lost by the use of the brakes, which includes the net grade resistance on the nearly level line. The train resistance of through passenger trains is estimated at 18 lb., that of the local trains at 26 lb. per ton. This includes the power wasted by the use of brakes. The last estimate is based on running sheets assuming an acceleration of 1.2 ft. per sec. and a retardation by the brakes of 2.4 ft. per sec.

Power is used for lighting the trains, for the operation of the brakes, and for switching. These last three items are estimated at 15% for freight trains, 7½% for through passenger trains and 5% for local trains.

If it is kept in mind that 1 h.p-day is equal to 23 760 ft-tons per day, the following formulas for the horse-power needed by the 37 daily freight trains averaging 1 600 tons, the 22 daily through passenger trains averaging 500 tons, and the 21 daily local passenger trains averaging 250 tons, each running over the 183 miles of line, are self-evident:

$$\frac{5\,280 \times 183}{23\,760} \times 37 \times \frac{1\,600 \times 8}{2\,000} \times 1.15 = 11\,073$$

$$\frac{5\,280 \times 183}{23\,760} \times 22 \times \frac{500 \times 18}{2\,000} \times 1.075 = 4\,328$$

$$\frac{5\,280 \times 183}{23\,760} \times 21 \times \frac{250 \times 26}{2\,000} \times 1.05 = 2\,914$$

$$18\,315$$

The tracks will be so crowded that the freight trains must run mainly during times of small passenger traffic. A load factor of two-thirds, therefore, can be easily secured. The maximum power required at the wheels of the motor units, therefore, will be $\frac{3}{2} \times 18\,315 = 27\,473$ h. p. The average efficiency of the electric locomotives and motor cars will be at least 70%; that of the low-tension transmission, at 15 000 volts, 96%; that of the step-down trans-

formers, 97 per cent. This gives an efficiency, from the primary of the step-down transformers to the wheels of the motor units, of $0.70 \times 0.96 \times 0.97 = 0.6518$ per cent. The maximum power required at the primary of the step-down transformers, therefore, is $\frac{27\,473}{0.6518} = 42\,150$ h. p.

The efficiency of the high-tension transmission is 85%, when the maximum power is transmitted. The total efficiency from the step-up transformer station to the wheels of the motor units, therefore, is $0.6518 \times 0.85 = 55.4$ per cent.

The maximum amount of power required at the secondary of the step-up transformers, therefore, is 49 600 h. p., or approximately, 50 000 h. p. At $12 per h. p. this will cost $600 000 per year.

Cost of Maintenance and Depreciation of Electric Locomotives, Motor and Passenger Cars.

The cost of maintenance and depreciation of the electric locomotives is not known. It is estimated at 10 cents per train-mile, or $3 500 per passenger locomotive and $2 500 per freight locomotive per year. This is probably excessive. The cost of maintenance and depreciation of motor cars is estimated at 8 cents per train-mile, or $1 867 per motor car per year. The cost of maintenance and depreciation of passenger cars is estimated at 1.4 cents per car-mile.

With steam-locomotive operation, approximately 224 cars go over the whole line daily. The future traffic will require 303 passenger-train cars to run daily over the whole line. Of these, 63 will be motor cars and 240 other passenger-train cars, an increase of 16. This represents 1 068 800 car-miles per year. The increase in cost of maintenance of passenger-train cars, exclusive of motor cars, therefore, is $15 000 per year. The cost of train service and motive power is estimated as follows: First, for the traffic with steam-locomotive operation.

Cost of Steam-Locomotive Power.—The Lake Shore Railroad report for 1905 gives the cost of locomotive power per locomotive-mile for the whole system as 32.07 cents. The revenue train-mileage is 15 901 016. The locomotive mileage is 25 741 488.

The cost of locomotive power per revenue train-mile, therefore,

is $\frac{25\,741\,488}{15\,901\,016} \times 32.07 = 55.28$ cents, say, 55 cents. The 28 passenger trains and 34 revenue freight trains per day represent 4 140 000 train-miles per year. The total yearly cost of locomotive power on the line and for the traffic here considered, therefore, is $2 277 000. The trains are larger on this line than the average train of the system; for this reason this is probably an underestimate.

Cost of Electric Locomotive and Motor Car Service.—For electric operations the cost of motormen for passenger trains is estimated at 3 cents, and for freight trains at 6 cents, per train-mile.

Cost of Train Service.—The other trainmen on the freight trains will cost considerably less with electric than with steam-locomotive operation, owing to the higher train speeds practicable with the former. The new passenger trains will have about one-third more cars running daily over the whole line and one-third greater speed; the cost of car service, therefore, will remain about the same.

Cost of Maintenance of Way.—In the opinion of high authorities, the cost of maintenance of way is mainly caused by the steam locomotives. The excessive weights on the drivers of the largest locomotives, especially at high speeds, are very severe on the track. The imperfectly balanced drivers and the heavy reciprocating parts produce vertical hammering and lateral swaying. The electric locomotives have no heavy reciprocating parts, and the weights on the drivers are very moderate. The cost of maintenance of way will probably be largely reduced. The amount of this reduction is unknown, and is here neglected.

Summary of Comparison of Costs.—The results of these considerations are summarized in the following tabulation:

Estimated future daily trains on 183 miles of line from Buffalo to Cleveland with steam-locomotive operation:

28 passenger trains of 8 cars,
34 revenue freight trains,
3 work trains.

Estimated future passenger earnings with steam-locomotive operation	$3 467 000
Estimated increase of passenger earnings by electric operation	1 330 000
Future passenger earnings, with electric operation	$4 797 000

Future trains with electric operation:

34 revenue freight trains,
3 work trains,
22 passenger trains of 9 cars,
21 passenger trains of 5 cars.

Cost of new plant for electric operation:

50 miles of tower lines	$ 100 000
5 100 000 lb. of copper for high-tension lines, in place, with switches, lightning protection, etc.	1 122 000
Overhead contact lines, sub-stations, and track bonding, etc., for 600 miles of track	2 880 000
50 miles of new track	1 000 000
	$5 102 000
42 passenger locomotives of 1 100 h. p. at $33 000	$1 386 000
90 freight locomotives of 1 000 h. p. at $30 000	2 700 000
60 motor cars of 500 h. p. at $18 000	1 080 000
	$5 166 000
150 steam locomotives and 103 passenger cars saved	2 286 000
Net cost of new rolling stock	2 880 000
Total	$7 982 000
Contingencies	798 000
Total new investment	$8 780 000

Cost of electric operation of future traffic:

Current, 50 000 h. p., at $12	$ 600 000
Maintenance and depreciation of 50 miles of tower lines, 3%	3 000
Maintenance of copper, etc., for high-tension lines, 1½%	16 800
Maintenance and depreciation of 600 miles of contact line and sub-stations, 4%	115 200

Maintenance and depreciation of electric locomotives, 10 cents per train-mile	394 100
Maintenance and depreciation of motor cars, 8 cents per train-mile	112 224
Motormen of freight trains, 6 cents per train-mile	148 296
Motormen of passenger trains, 3 cents per train-mile	86 170
Increase in cost of maintenance of passenger cars	15 000
Total	$1 490 790
Interest and taxes on cost of plant, 5%	439 000
	$1 929 790
Cost of steam-locomotive power saved	2 277 000
Net saving	$ 347 210
Increase in passenger earnings	1 330 000
Total gain	$1 677 210

This is 19.1% of the capital invested, and is in addition to about 4% interest counted among the yearly expenses. If the power were derived from four steam-turbine power-stations along the line, the stations with their machinery would cost about $3 200 000. The transformer sub-stations could be replaced by feeders. The power could be produced at the stations in the form of 15 000- or 20 000-volt, single-phase current. The saving in the cost of the transmission system would be about $1 200 000. The net increase in investment would be about $2 000 000.

The efficiency of transmission would be about 95%, and the total efficiency, from power-stations to wheels, $0.70 \times 0.95 = 66.5$ per cent. The average power required at the power-houses would be approximately 20 600 kw., its cost at the power-house would be approximately $60 per kw. per year, or $1 236 000, allowing 5% on the cost of the plant for interest and taxes.

The cost of water-power and of its transmission, amounting, with interest and taxes on transmission plant, to $680 900, would be saved. The net increase in yearly cost of operation and interest and taxes, therefore, would be $555 100. This would reduce the

yearly gain from electric operation to $1 142 200, or 10.6% on the investment. Adding 4%, as previously allowed, the total return on the invested capital would be 14.6 per cent. With the more expensive steam power, better financial results would probably be obtained with slower passenger trains, requiring much less power. It is apparent, therefore, that the electric operation of lines of similar amount and kind of traffic will be profitable even where no water-power is available. The density of population along the line, the degree of its prosperity, the presence or absence of a competing electric line, and the fare per passenger-mile determine the increase in local passenger traffic that can be counted upon. The amount of existing through passenger traffic on the line considered, and the intensity of competition for it, as well as the amount of the through passenger traffic on the competing lines and the fare per passenger-mile, determine the increase in through passenger traffic which will be the result of electric operation and the consequent improvement in the service. On all lines of large traffic, the increase in traffic capacity of a given number of tracks will give very valuable advantages which have been entirely neglected in this calculation.

Alternate Estimate.

The adopted estimates of the increase of the passenger earnings are based on voluminous statistics of the amounts of passenger earnings secured by the interurban electric roads. These statistics can be found in various technical and financial periodicals of the last few years.

The writer was greatly surprised at the amount of new passenger traffic developed by these roads. Without knowledge of the results obtained by them, he would have considered excessive the estimates here given for the probable increase of earnings due to the introduction of electric operation. It would probably require a longer paper than that here submitted to convince those unfamiliar with the facts here referred to that the estimate herein adopted is conservative and will be more than confirmed by the actual results. Since this estimate will be accepted as reasonable by but few without the elaborate proofs which cannot here be given, it is desirable to show how a lower estimate of the increase in earnings will affect the rate of return on the invested capital.

Where Niagara Falls or other equally cheap power is obtainable it can be easily shown that the electric operation of a line with the traffic here assumed is advisable even with one-half the increase in passenger earnings above estimated. The reduction of $665 000 in passenger earnings, as compared with the foregoing estimate, would permit a reduction of one in the average number of cars per passenger train. The locomotives for the through passenger trains would then have to be of 1 000 h. p., and the local passenger trains would require, with a slight reduction of the schedule speeds, an average of 2 motor cars instead of 3. This would reduce the power required to 94% of the former amount. An equal reduction would take place in the required capacity of the transformers, and in the amount of copper in the high-tension lines.

The number of motor cars required would be reduced to 40. The reduction in cost of plant would be approximately as follows:

Copper for high-tension line	$ 67 300
Transformers	21 600
Motor cars	360 000
Electric locomotives	126 000
20 passenger cars	80 000
	$654 900
Contingencies 10%	65 500
Total reduction of cost of plant	$720 400

This gives the revised cost of plant as $8 059 600.

The reduction in yearly cost of operation would be as follows:

Current	$ 36 000
Maintenance of copper	1 000
Maintenance of sub-stations	1 000
Maintenance and depreciation of motor cars	37 400
Maintenance of passenger cars	20 570
Interest and taxes on cost of plant	36 020
Increase in net saving	$131 990

The new net saving by electric operation would be $479 200. If $665 000 is added, for the new increase of passenger earnings, the result will be $1 144 200 as the total gain by electric operation. This is 14.2% of the total investment. The total return on invested capital, inclusive of about 4% interest allowed among the operating expenses, therefore, is 18.2 per cent. This estimate will undoubtedly appear more reasonable to those unacquainted with recent results on interurban electric roads.

Cost of Hauling a Car.—It is of interest to compare the cost of hauling a freight or passenger car 1 mile by steam locomotive and by electric power. In this calculation it is fair to include the interest and taxes on the cost of the plant, because the stockholders of the corporation, who will govern the decision, are mainly interested in the question whether the cost of hauling, inclusive of the new interest charges, is increased or decreased.

It is found, by distributing the total cost of locomotive power among the number of revenue train-miles, that the cost per revenue train-mile is approximately 55 cents. If this cost is assumed to be the same for passenger and freight trains, there is obtained, with 8 cars per passenger train, 6.875 cents, as the cost per car-mile. This does not include the interest and the taxes on the value of the locomotive and tender. Taking their value to be $12 500, and estimating, from published data, the number of revenue passenger train-miles per locomotive at 33 000 per year, and taking interest and taxes at 5%, the interest charge per car-mile is equal to $\frac{12\,500 \times 0.05}{33\,000 \times 8} = 0.237$ cents. This gives for the total cost of hauling a passenger car 1 mile in a revenue train by steam locomotive $6.875 + 0.237 = 7.112$ cents. This includes the estimated share of the cost of switching and work trains.

The freight cars at present in use vary considerably in size. The writer has calculated the cost of hauling 1 mile a car weighing 70 tons when fully loaded, inclusive of the estimated share of the cost of the switching and work trains.

For the average train of 1 600 tons, the locomotive and tender will weigh approximately 150 tons. The cost of hauling a train 1 mile is 55 cents, exclusive of interest and taxes. Taking 23 000 revenue freight train-miles per locomotive per year, the interest

and taxes per train-mile amount to $\frac{12\,500 \times 0.05}{23\,000} = 2.72$ cents; this makes the total cost of hauling a freight train 1 mile, including a proper share of the cost of the switching and work trains, 57.71 cents. The average train of 1 600 tons, inclusive of locomotive, contains 20.71 cars of 70 tons. The cost per car-mile, therefore, is $\frac{57.71}{20.71} = 2.788$ cents.

Cost of Electric Haulage.—For determining the cost of electric power at the motor car or electric locomotive, there must be added to the cost at the power-house that of transmission. The cost of the transmission plant proportioned for receiving 50 000 h. p. and delivering at motor cars 39 575 h. p. is $4 102 000. The total cost of the delivered power, therefore, is as follows:

50 000 h. p. at power station.....	$600 000
5% interest and taxes.....	205 100
Maintenance of copper, etc.....	16 800
Maintenance of 50 miles of tower line..	3 000
Maintenance of secondary lines and transformer stations	115 200
Total cost at train of 39 575 h. p.....	$940 100

Cost per horse-power per year, $23.75.

Since the load factor is two-thirds, the horse-power-year contains $\frac{2}{3}$ of 8 760 h.p-hr. The cost of power per horse-power-hour used at motor car or locomotive, therefore, is 0.4067 cents.

The power required at the wheels for hauling 22 daily through passenger trains a distance of 183 miles was found to be 4 328 h. p. This includes the power required for switching, train lighting and the operation of the brakes. It does not include the power for the three work trains. The latter is $\frac{3}{37}$ of 11 073 = 898 h. p. In the calculation of the cost of steam power, all the costs of power have been charged to the revenue trains. Following the same method, the cost of the power for the work trains will be charged to the cost of hauling the passenger trains. The cost of power per passenger train-mile becomes, therefore, $\frac{(4\,328 + 898) \times 24}{22 \times 183 \times 0.7} \times 0.4067$ = 18.11 cents. These trains have an average of 9 cars; the cost of

power per car-mile, therefore, is 2.012 cents. The costs incurred on the locomotive, per revenue train-mile, are 10 cents for maintenance and depreciation, 3 cents for motormen, 4.73 cents for interest and taxes, a total of 17.73 cents, or 1.97 cents per car-mile. This gives the total cost of hauling a passenger car 1 mile 2.012 + 197 = 3.982 cents. The schedule speed of this car is one-third greater than that of the car hauled by the steam locomotive. This higher speed increases the energy required about 25 per cent. The corresponding cost of steam-locomotive haulage at the slower speed was found to be 7.112 cents. Steam-locomotive haulage at the higher speed, if practicable at all, would cost more than twice as much as electric haulage.

It was found that it takes 11 073 — 898 = 10 175 h. p. to haul 34 daily revenue freight trains a distance of 183 miles. The cost of power per revenue train-mile, therefore, is $\frac{10\,175 \times 24}{34 \times 183 \times 0.7} \times 0.4067$ = 22.81 cents.

The costs per train-mile on the locomotive are:

Maintenance and depreciation, 10 cents; motormen, 6 cents; interest and taxes, 6 cents; total 22 cents.

The cost of hauling a 1 600-ton freight train 1 mile, therefore, is 22.81 + 22 = 44.81 cents. This is $\frac{44.81}{20.71}$ = 2.164 cents per car-mile. This includes the cost of switching and the operation of the brakes.

The speed is one-third greater than with steam locomotives; this reduces the cost of the train crews per train-mile. The corresponding cost of steam haulage at the slower speed is 2.788 cents. In the same manner the cost of hauling with electric power a passenger car of a 5-car train is found to be 5.09 cents. The schedule speed is one-third greater than for steam trains with the same number of stops. All items of cost that are the same with steam locomotives and by electric operation have been omitted in these comparative estimates.

Conclusions.

Unless these calculations are greatly in error, it is evident that the electric operation of many steam railroad lines will be extremely

profitable, especially where the passenger traffic is large, where there are no existing parallel electric lines, and where cheap water-powers are available.

The writer is indebted to the General Electric Company for information in regard to the cost of single-phase motor cars and locomotives. This information, however, was received two years ago, and the prices given in this paper are based mainly on later published prices. All estimates of cost of electric plant and operation were intentionally taken very high. Future experience alone can determine the accurate costs of electric operation. The writer is convinced that the cost of electric operation here given is in excess of what it will prove to be upon actual trial. The figures given were chosen with a view of being able to prove from past experience with similar, though not identical, apparatus and plant that they are at least adequate. The cost of single-phase electric locomotives and motor cars will probably greatly decrease within a short time, and this will make possible a much better showing for electric operation.

For roads with small traffic, an entirely different and much cheaper contact line, and smaller and slower trains would be used. The results here obtained do not justify any inference either for or against the electric operation of such roads.

The arguments given in this paper are the result of the condensation and revision of several voluminous reports, by the writer, on this subject. After discussion, further details will be given on points which are contested, if the writer is able to do so.

Various estimates of the cost of electrically operating a given amount of either freight or passenger traffic have been published. The writer believes that the only way to obtain a correct answer to the question, Will electric operation pay? is to consider the problem as a whole, and to compare the probable increase in future net earnings, due to the changes and improvements in passenger service which become practicable and will be adopted with the introduction of electric operation, with the required new investment of capital. Wherever electric operation has been adopted, the gain in passenger earnings has been by far the most important factor in making the change profitable. The writer hopes that the discussion will bring forth valuable new information on various parts of the subject.

APPENDIX.

THE MECHANICAL THEORY OF THE CONTACT CONDUCTOR FOR HIGH-SPEED TRAINS.

No contact conductor is absolutely straight when the sliding bow passes over it. The latter, therefore, does not move in a straight line, but rises and falls alternately. There are two methods of keeping the contact conductor approximately straight. One consists in using a rod, of sufficient cross-section to resist the bending stresses to which it is exposed and avoid temperature stresses by means of expansion joints. The other method consists in using a wire under tension. This wire will be pushed up by the pressure of the sliding bow and then pulls down on both sides of it. If it is large, and especially if it is grooved and has an elongated cross-section, it has also considerable stiffness and resists some part of the pressure of the sliding bow by acting as a girder. The tension in the wire will vary greatly according to the temperature. The modulus of elasticity of a copper wire is about 20 000 000 lb. per sq. in.; the coefficient of expansion is 1 in 104 400 per degree Fahrenheit. If the temperature varies 140° there is $\frac{140}{104\,400} \times 20\,000\,000 = 26\,820$ lb. change in tension per sq. in. if the length of the wire remains the same. If the wire has always the same length, excessive strains at minimum temperatures can only be avoided by making the tension at maximum temperatures very small. With rope suspension and 300-ft. spans, the deflection of the wire, with changes of temperature, varies only about 1 ft. The length of the wire changes so little that this change is quite negligible, if the wire is taut in summer. This practically constant length of the contact wire is one of the serious drawbacks of rope suspension, as it greatly increases the variation in tension due to changes of temperature.

If the sliding bow moves rapidly up and down, its pressure on the contact conductor will vary greatly. It may become so great as to break the conductor, or may become zero and cause a separation accompanied by sparking. An investigation of these variations of pressure between sliding bow and contact conductor, therefore, is essential in making a rational design of either. The calculation is somewhat simpler with a contact rod acting only as a girder than with a wire acting as a tensile member and as a girder. For this reason the writer will begin with the former.

It will be assumed that a soft steel rod, $1\frac{1}{8}$ in. in diameter, is suspended from a rope by suspenders 15 ft. apart. This rod, be-

cause of its weight, will not hang straight, but in a waved line. The equation of its curve between two succeeding suspenders is:

$$y = \frac{P}{E\,I}\,\frac{l^3}{24}\left\{\frac{x^2}{l^2} - \frac{2\,x^3}{l^3} + \frac{x^4}{l^4}\right\}.$$

In this equation,

P is the weight of one panel of rod = 50 lb.;

E is the modulus of elasticity of the material = 29 000 000 lb. per sq. in.;

I is the moment of inertia of the cross-section of the rod = 0.07862 in.;

l is the length of the panel = 180 in.

The origin of co-ordinates is at the intersection of the suspender with the rod. First, it will be assumed that the sliding bow or roller which collects the current moves along this curve with a velocity of 80 miles an hour = 1 408 in. per sec., and the variation of pressure between contact rod and oscillating sliding bow will be investigated. If the time begins when the sliding bow passes a suspender, and if x designates the distance of the point of contact from the same suspender, then $x = v\,t$, where v is the velocity, in inches per second, and t the time, in seconds. The current collector moves on the curve given by the equation:

$$y = \frac{P}{E\,I}\,\frac{l^3}{24}\left\{\frac{x^2}{l^2} - \frac{2x^3}{l^3} + \frac{x^4}{l^4}\right\}.$$

If x is replaced by $v\,t$, then

$$y = c\left\{\frac{v^2\,t^2}{l^2} - \frac{2\,v^3\,t^3}{l^3} + \frac{v^4\,t^4}{l^4}\right\}, \text{ where } c = \frac{P}{E\,I}\,\frac{l^3}{24}.$$

Differentiating the equation twice gives:

$$\frac{d\,y}{d\,t} = c\left\{\frac{2\,v^2\,t}{l^2} - \frac{6\,v^3}{l^3}\,t^2 + \frac{4\,v^4}{l^4}\,t^3\right\}$$

$$\frac{d^2\,y}{d\,t^2} = c\left\{\frac{2\,v^2}{l^2} - \frac{12\,v^3}{l^3}\,t + \frac{12\,v^4}{l^4}\,t^2\right\}.$$

This gives, for $t = 0$, at the end of the span,

$$\frac{d^2\,y}{d\,t^2} = \frac{c\,2\,v^2}{l^2} = \frac{P}{E\,I}\,\frac{l}{12}\,v^2.$$

For $t = \frac{l}{2\,v}$ at the center of the span,

$$\frac{d^2\,y}{d\,t^2} = -\frac{c\,v^2}{l^2} = -\frac{P}{E\,I}\,\frac{l}{24}\,v^2$$

The expression, $\frac{d^2\,y}{d\,t^2}$, is evidently the acceleration of the vertical motion of the contact point of the current collector. This acceleration is produced by the resultant of the forces acting upon it. These forces are the downward pressure of the contact conductor,

the weight of current collector, and the upward pressure of the springs supporting the latter. The collector itself may be a spring of which only the part at and near the contact point moves up and down, so that the weight moving up and down is small; or it may be a roller, of copper or brass, which is rigid and moves up and down as a whole. The ends of the roller must be supported on springs for the purpose of reducing the weight moving up and down once in each span.

Past experience on trolley lines and on the Valtellina Railroad is strongly in favor of a rolling contact between the wire and the collector. The roller, however, is not practicable, as will be shown in the following, where the highest speeds and a contact wire carried by frequent suspenders are used.

With a contact rod a sufficiently light roller can easily be designed. If a roller having a weight of w pounds, is assumed to be supported on springs sufficiently yielding not to transmit the rapid up and down oscillations to the frame below, then the force needed to produce the acceleration given above, for $t = 0$, is

$$F = \frac{w}{12 \times 32.2} \frac{d^2 y}{d t^2} = \frac{w}{12 \times 32.2} \frac{P}{E I} \frac{l}{12} v^2.$$

If $w = 25$ lb., and the values of P, l, E, I and v be the same as given above, all dimensions being in inches, then

$$F = \frac{25}{12 \times 32.2} \times \frac{50 \times 15 \times 1\,408^2}{29\,000\,000 \times 0.07862}, \text{ or}$$

$F = 42.19$ lb. For $t = \frac{l}{2 v}$, which corresponds to the center of the span, $F = -21.1$ lb.

The meaning of these results is that at the suspenders, where the curve of the contact rod is convex above, the collector presses upward not only with the pressure which it exerts when it moves in a straight line, or is at rest, but with this pressure increased by 42.19 lb., due to its curved motion. At the center of the 15-ft. span the curve of the contact rod is convex downward. The pressure of the roller is reduced by 21.1 lb., due to the curvature of the rod. To avoid the separation of the collector and the contact rod, the pressure when at rest must be at least 21.1 lb. To allow for inaccuracy and leave a moderate minimum pressure to produce good contact, it should be somewhat larger, say 25 lb.

The radius of curvature of the rod at any point is approximately equal to $\frac{1}{\frac{d^2 y}{d x^2}}$. This gives, for the radius at the suspender, from the above equation of the curve, $r = 253$ ft., and for the center of the span, $r = 506$ ft. The variation of pressure between

the sliding bow and the contact rod might also be considered as the variation of the centrifugal force during the motion in a curve. The centrifugal force is

$$F = \frac{w}{g}\frac{v^2}{r}, \quad r = \frac{1}{\frac{d^2 y}{d x^2}}.$$

Therefore, $F = \frac{w}{g}\frac{d^2 y}{d x^2} v^2$. If $\frac{d^2 y}{d x^2}$ is obtained from the above equation of the curve, it is seen that this formula is identical with the previous one.

The deflection of the curve at the center of the span is $\frac{1}{8}$ in.

From these results, it is evident that at the high speeds here assumed an apparently trifling deviation from a straight line motion produces a large variation in pressure on the contact conductor.

If a roller weighing 25 lb. is used as a current collector, its pressure on the contact conductor when at rest should be 25 lb., if it is certainly to touch the conductor at the center of each span. The pressure, when it is at the suspenders, is then $25 + 41.2 = 66.2$ lb. With the rod assumed, this would produce compression in the suspenders and would also produce larger strains than desirable in the rod. From the formula for F, it is clear that doubling the velocity means a four-fold increase in the variation of pressure on the contact rod, due to velocity. In the foregoing it has been assumed that the curve of the contact rod remains unchanged when the collector passes. The collector, when at the center of the span, exerts only a trifling pressure on the rod. This pressure slightly diminishes the deflection and curvature, and improves the contact. When the collector is at or near a suspender it exerts a considerable pressure, but it diminishes the tension in the suspender nearly the same amount; this largely neutralizes its effect on the curvature. The error in the foregoing results, therefore, is small.

The proper size of the contact rod evidently depends on the maximum train speed and on the oscillating weight of the collector. With a speed of 80 miles an hour, and a rigid collector weighing 25 lb. above the supporting springs, a rod $1\frac{1}{4}$ in. in diameter is advisable.

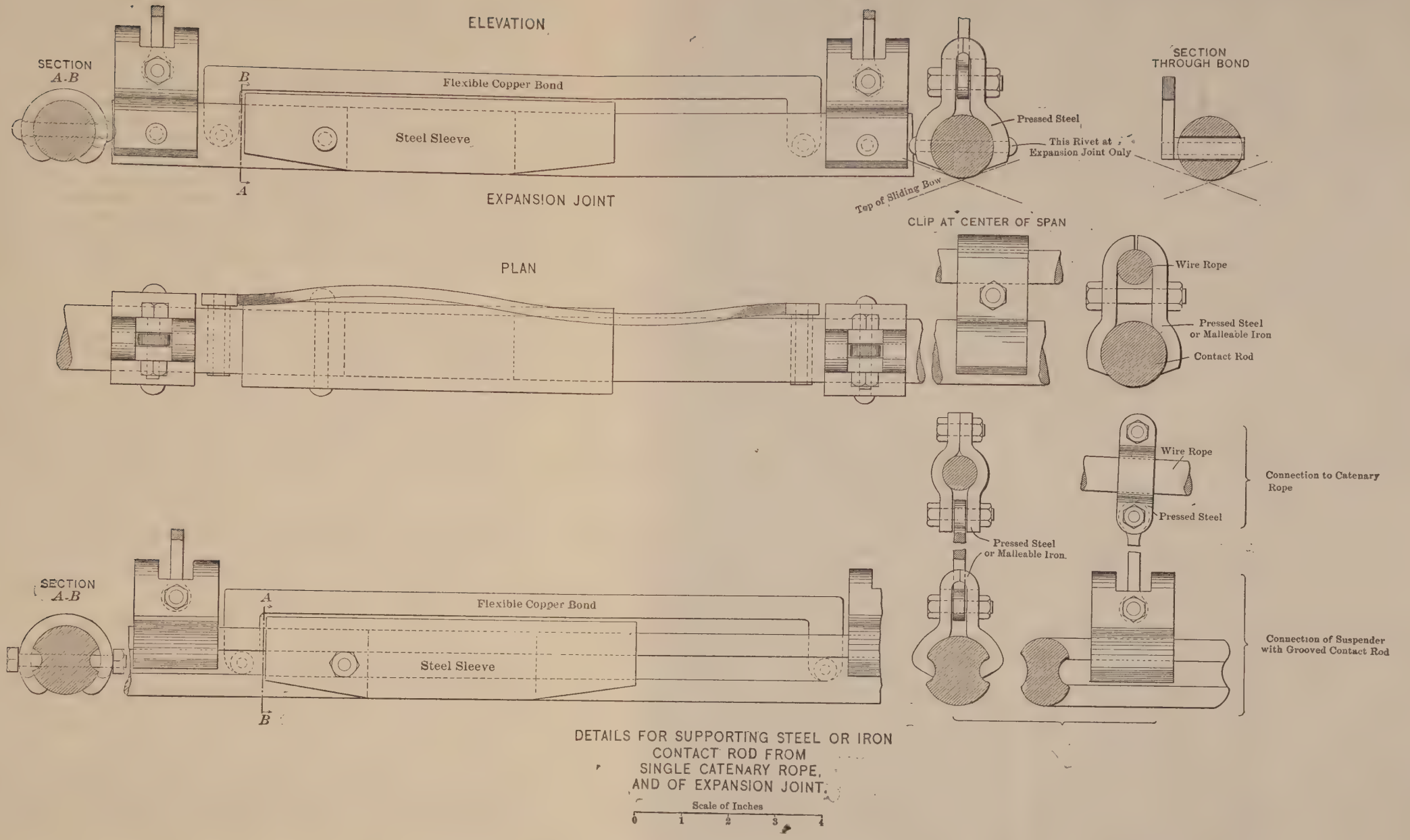
With this rod, the suspenders can be spaced 12 ft. apart without risk of compression when the sliding bow passes. Calculation shows that the center deflection of such a rod is only $\frac{1}{5}$ in. The up and down oscillation of the collector increases the pressure on the contact conductor 22 lb. when it passes the suspender, and it decreases the pressure 11 lb. when it is at the center of the span. The pressure when at rest may be made 20 lb. The maximum pressure will then be 42 lb., and the minimum 9 lb. The largest bending

strain per square inch in the rod will be so small that it will be safe even with welds of mediocre quality. The collector roller might be considerably heavier than 25 lb. without risk of breaking the contact rod. With a 1¼-in. contact rod, therefore, it is easy to design a rolling collector which has abundant strength and is very durable, thus greatly diminishing the cost of maintenance.

A contact rod with an expansion joint and suspender clips is shown in Plate XLI.

Contact Wire.

A contact wire under tension, with suspenders 10 ft. apart, is practically straight. When the sliding bow comes along, it is bent up. The deflection varies with the pressure of the sliding bow and the tension in the wire, in short spans, also inversely with its stiffness. With 140° fahr. variation of temperature, the tension is 26 820 lb. per sq. in. more at the minimum than at the maximum temperature. The total strain in the wire from all causes should never reach the elastic limit. The elastic limit of a hard-drawn 0000 copper wire is about 35 000 lb. per sq. in. It has previously been seen that a very small deflection of the contact conductor with very even curves produces a large variation in the pressure between the current collector and the contact conductor. A deflection of $\frac{1}{9}$ in. at the center of a 12-ft. span produced a variation of 33 lb. in the pressure of a collector weighing 25 lb. This variation in pressure arises from, and is proportional to, the sum of the maximum acceleration and retardation of the vertical motion of the collector. Assuming 10-ft. spans between two succeeding suspenders of the contact wire, the time of one oscillation of the vertical motion is ten-twelfths of the time with the above considered 1¼-in. contact rod of 12-ft. span. From the general formula for uniform accelerated motion, $s = \frac{g}{2} t^2$, or $g = \frac{2\,s}{t^2}$, where s is the space covered, g the acceleration, and t the time, it follows that, other things being equal, the acceleration is inversely proportional to the square of the time within which a certain distance is covered. If the uniformity of the acceleration and retardation and the amplitude of the vertical motion are the same, then 10-ft. spans require 1.44 times the maximum acceleration of 12-ft. spans. The maximum acceleration with a given uniformity of motion is also proportional to s, or, in this case, the amplitude of the vertical motion. The variation of the contact pressure is proportional to the maximum acceleration multiplied by the oscillating weight. The oscillating weight can be reduced by using a sliding bow which is a spring. Only a part of its weight will then move up and down with the lower face of the wire. It will be assumed that it is practicable to reduce in this



DETAILS FOR SUPPORTING STEEL OR IRON CONTACT ROD FROM SINGLE CATENARY ROPE, AND OF EXPANSION JOINT.

Scale of Inches

0 1 2 3 4

manner the effective oscillating weight to 6 lb. To obtain a variation of pressure of 33 lb. between the current collector and the contact conductor, the same as with the 1¼-in. rod of 12-ft. span, the amplitude of motion should be $s = \frac{1}{9} \times \frac{25}{6} \times \frac{1}{1.44} = 0.320$ in.

Another course of reasoning for determining the relation between the variation of the contact pressure and the amplitude of the vertical oscillation of the sliding bow is as follows:

The sliding bow presses up with a static pressure, Z_s. The contact wire cannot balance this pressure without upward deflection. The sliding bow, therefore, rises between the suspenders and falls at the suspenders. It moves up and down with an accelerated or retarded motion. If the upward acceleration is α, if w is the weight of the oscillating part of the sliding bow, if g is the acceleration of gravity, and if Z_d is the varying dynamic pressure of the sliding bow, then $Z_d = Z_s - \alpha \frac{w}{g}$.

The largest positive and the largest negative value attained by α determine, with a given train speed and distance of the suspenders and the oscillating weight, the amount of variation of the contact pressure. For a given amplitude of oscillation, α would be smallest if it were constant during the acceleration and constant during the retardation. The oscillation would then consist of a uniformly accelerated upward motion followed by a uniformly retarded upward motion; these would be followed by a uniformly accelerated and a uniformly retarded downward motion. The upward motion would take half the time of one oscillation, the downward motion the other half.

The accelerated upward motion would be one-half, the retarded upward motion would be the other half, of the total upward motion.

The total variation of pressure would be $2 \alpha \frac{w}{g}$. If the variation of pressure be taken as 33 lb., and $w = 6$ lb., then $\frac{2 \alpha \times 6}{12 \times 32.2} = 33$, and $\alpha = 1\,062.6$ in.

The amplitude of the oscillation would be the distance covered during the accelerated, plus that covered during the retarded, upward motion. The time for each of these is $t = \frac{0.0852}{4} = 0.0213$ sec. The amplitude, $A = \alpha t^2 = 0.4824$ in. In reality, the acceleration during the accelerated upward motion varies from a maximum to 0, and the retardation increases from 0 to a maximum. It is evident, therefore, that the amplitude, 0.4824, is much larger than the actual one. The assumption that the acceleration uniformly

decreases from the beginning to the end of the upward motion, being at the end of upward motion negative and equal to the acceleration at the beginning, therefore corresponds more closely to the facts than the assumption above made.

In accordance with this new assumption, $\alpha_{max.} = 1\ 062.6$ in. and $\alpha = 1\ 062.6 - c\ t$, $t = 0.0426$, corresponds to the end of the upward motion. Then, $-1\ 062.6 = 1\ 062.6 - c \times 0.0426$, or $c = 49\ 880$. Therefore, $\alpha = \frac{d^2\ y}{d\ t^2} = 1\ 062.6 - 49\ 880\ t$.

By integrating:

$$\frac{d\ y}{d\ t} = 1\ 062.6\ t - 24\ 940\ t^2 + c,$$

for $t = 0, \frac{d\ y}{d\ t} = 0,$

therefore $c = 0.$

By integrating again:

$$y = 531.3\ t^2 - 8\ 313.3\ t^3 + c,$$

for $t = 0, y = 0,$

therefore $c = 0.$

For $t = 0.0426$, $y = 0.3215$ in. which is the amplitude of oscillation. The value previously found, namely, 0.320 in., is confirmed, therefore, as at least approximately correct.

The minimum tension in the wire which will limit the amplitude of oscillation to 0.320 in. must now be determined. The static pressure of the sliding bow will be assumed to be 20 lb., and the oscillating weight 6 lb. If the sliding bow were not opposed during the upward motion, which is assumed to cover half the time of one oscillation, it would rise a distance, $s = \frac{2032.2}{6 \times 2} \times 12 \times 0.0426^2 = 1.17$ in. The opposing pressure of the wire must reduce this to 0.320 in.

The opposing pressure of the wire is due mainly to its stiffness and its tension.

Assume a 0000, B. & S. gauge, hard-drawn, copper wire of elongated cross-section, equivalent to two 0 wires touching and connected at contact. Such a wire has a depth of cross-section of 0.65 in. and a moment of inertia, around the short axis through the center of the cross-section, of 0.005476 in., and, consequently, a moment of resistance of 0.01685 in. If such a wire with free ends is exposed to a deflecting force, P_b, acting at a distance, c, from one end and c_1 from the other end of a 120-in. span, the equation for its deflections at the point of application, of P_b is $f_b = \frac{P_b\ c^2 c_1^2}{El\ 3l}$.

Introducing the foregoing values of E, I and l, gives

$$f_b = \frac{P_b\, c^2\, {c_1}^2}{39\,427\,200}; \text{ or } P_b = \frac{39\,427\,200}{c^2\, {c_1}^2} f_b.$$

If a flexible wire is exposed to a tension, T, and is deflected by a force, P_t, acting at the distance, c, from one end of the span, as shown in Fig. 1, then, for small deflections, approximately:

$$\left\{\frac{f_t}{c} + \frac{f_t}{c_1}\right\} T = P_t,$$

$$\frac{c + c_1}{c\ c_1} f_t\, T = P_t,$$

$$c + c_1 = l,$$

therefore,

$$P_t = \frac{l}{c\ c_1} f_t\, T.$$

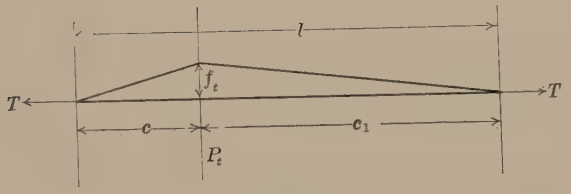


FIG. 1.

The total resistance opposed to deflection by a stiff wire under tension is approximately $P_b + P_t$. Table 1 gives, for different values of c and for a deflection of 0.1 in., the values of $P_b + P_t$ calculated by the foregoing formulas for a tension, $T = 1\,000$ lb.

TABLE 1.

$c = 0.1\ l$;	$P_b + P_t$	= 2.348 + 9.260	=	11.608 lb.
$c = 0.2\ l$;	$P_b + P_t$	= 0.743 + 5.208	=	5.951 "
$c = 0.25\ l$;	$P_b + P_t$	= 0.541 + 4.444	=	4.985 "
$c = 0.30\ l$;	$P_b + P_t$	= 0.431 + 3.968	=	4.399 "
$c = 0.40\ l$;	$P_b + P_t$	= 0.330 + 3.472	=	3.802 "
$c = 0.50\ l$;	$P_b + P_t$	= 0.304 + 3.333	=	3.637 "

Assuming the uniform motion most favorable to a small variation of the contact pressure, then the minimum pressure is 16.5 lb. less than the static pressure of 20 lb., or equal to 3.5 lb. The upward motion occurs approximately at the center half of the span; its beginning is about when $c = 0.25\ l$. The deflection of the wire is then $0.1 \times \frac{3.5}{4.985} = 0.07$ in. If the amplitude of oscillation is 0.320 in., the deflection at the end of the upward motion is 0.39 in. With this deflection, and with $c = 0.75\ l$

(which gives the same pressure as $c = 0.25\ l$) the opposing pressure is $4.985 \times 3.9 = 19.44$ lb. The opposing pressure should be $33 + 3.5 = 36.5$ lb. With the tension assumed, the amplitude of the oscillation would be larger. It would be about 0.67 in. if the variation of pressure did not increase with the larger amplitude. This result indicates that the tension assumed is not adequate to prevent the jumping of the sliding bow. The upward motion, however, may take place not quite at the center half of the span. If it begins with $c = 0.3\ l$ and ends with $c = 0.8\ l$ the deflection of the wire at the beginning is $0.1 \times \frac{3.5}{4.4} = 0.08$. $0.320 + 0.08 = 0.4$, and the opposing pressure at the end is $4 \times 5.951 = 23.804$. This opposing pressure is also quite inadequate.

If the upward motion did occur between $c = 0.4$ and $c = 0.9$, then the opposing pressure at the end of the upward motion would be ample. It is evident, however, that the upward motion will occur near the middle half of the span, where the resistance opposing it is smallest. The writer believes that the above reasons amply justify the statement that a tension in the wire of less than 1 000 lb. is certainly inadequate to prevent the jumping of the sliding bow.

Increasing the static pressure of the bow is no remedy for the evil, as this would increase the deflections or the amplitude of the vertical oscillation, consequently the maximum acceleration and retardation and the variation of the contact pressure. The only remedies are a diminution of the oscillating weight or an increase of the tension in the wire. The various small inaccuracies in the foregoing calculation are not great enough to vitiate the above inference because the deficiency in the opposing pressure of the wire is very large. The writer has assumed that it is practicable, by a spring support of the wire at the suspender, to regulate the vertical oscillation of the sliding bow so that the up and down motions each take one-half the time of one oscillation. If the oscillation is irregular, which is probably unavoidable, then the maximum acceleration and retardation during the more rapid part of the oscillation will be larger than estimated above, the variation of contact pressure will be greater, and a greater tension in the wire will be required to prevent jumping and consequent sparking. The above minimum tension of 1 000 lb. is 6 000 lb. per sq. in. of wire. The maximum tension per square inch at the lowest temperature is $6\,000 + 26\,820 = 32\,820$ lb. This will be increased by the normal pressure of the passing sliding bow.

The writer will now consider the effect of the pressure of the sliding bow on a wire under great tension. The wire will be bent up. If T is the tension and P is the deflecting force, then the wire, near P, will be bent in a curve.

If the angle of the two end tangents is α, then $\alpha = \frac{P}{T}$ for small values of P. The bending moment at any point, A, is equal to $T \times A a$, or T multiplied by the distance of the end tangent from the point, A, of the curve. At the point of application of P, the bending moment is largest, and is equal to $c\ D \times T$. If the height of the cross-section of wire is h, and if r is the radius of curvature of the wire at any point, then the bending strain per square inch in the extreme fiber is $s = \frac{h}{2\ r}$ 20 000 000 lb., where 20 000 000 lb. is the modulus of elasticity of copper wire. $h = 0.65$ in.

Therefore,

$$s = \frac{6\ 500\ 000 \text{ lb.}}{r}$$

Also, $s = \frac{M}{R}$, where M is the bending moment of the external forces and R is the moment of resistance of the cross-section of the wire. From these equations, $r = \frac{6\ 500\ 000}{M} R$.

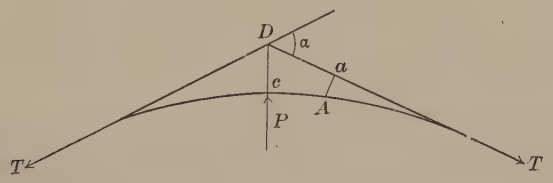


FIG. 2.

The value of $c\ D$ corresponding to a given value of α would be smallest if the maximum curvature extended over the whole length of the curve or if the curve were a circle. In this case, $c\ D$ would be equal to $\frac{r\ \alpha^2}{8}$, and the bending moment at the contact point of the sliding bow would be $M = \frac{r\ \alpha^2}{8} T$, or, introducing for α its value, $\frac{P}{T}$, then $M = \frac{r\ P^2}{8\ T}$. Introducing the above value of r, gives $M = \frac{812\ 500\ P^2}{M\ T} R$, or $M = P \sqrt{\frac{812\ 500\ R}{T}}$, and $s = \frac{M}{R} = \frac{P \times 901.4}{\sqrt{R\ T}}$.

The maximum value of $T = 32\ 820$ lb. per sq. in., or $T = 5\ 470$ lb. $R = 0.01685$ in. P is the maximum pressure of the sliding bow This will be assumed to be 24 lb. Then, to obtain s:

$$s = \frac{24 \times 901.4}{\sqrt{5\ 470 \times 0.01685}} = 2\ 253 \text{ lb.}$$

This value of s is not the actual value, but the value corresponding to a circular curve. The curve actually straightens out toward the ends. The length of the curve corresponding to the angle, α, and the distance, c D, therefore, is greater than assumed. From this it is known that the bending strain in the wire, per square inch of extreme fiber, is considerably more than 2 253 lb.

When there is a wind, the wire is blown to leeward, and the long axis of the cross-section becomes inclined instead of vertical. The upward pressure of the sliding bow will twist the wire when the contact point is near the center of the span so that the long axis becomes still more inclined. If it were horizontal, the bending strain would be 1.58 times as large as above calculated.

The bending strain must be added to 32 820 lb. to obtain the total strain. The sum exceeds the elastic limit of the wire.

In view of the fact that breaks of trolley wires of much smaller size seldom occur, it is important to bear in mind that the variation of the pressure of the current collector, and consequently its maximum pressure, varies approximately as the square of the frequency of oscillation. The frequency of oscillation varies as the train speed, and inversely with the span. The deflection caused by the sliding bow varies approximately as the span and as the pressure of the bow. Short spans between the suspenders of the contact conductor and especially high train speeds are very unfavorable. Any favorable inference drawn from the experience on trolley lines, therefore, is not well founded, since the conditions are demonstrably vastly different. Rope suspension is responsible for an increase in the variation of tension of the contact wire due to changes of temperature. This variation is the most important cause of trouble. The high voltage used demands absolute safety; the safety secured by an ordinary trolley line, therefore, is inadequate and should be improved.

Summary.

The result of this investigation may be summed up as follows: A 1½-in. contact rod suspended from one rope hung in 300-ft. spans, by suspenders 12 ft. apart, sags only $\frac{1}{9}$ in. at the center of each 12-ft. span. The current collector, if moving with a speed of 80 miles an hour, will oscillate up and down about $\frac{1}{9}$ in. while passing under a 12-ft. span. This oscillation produces with an oscillating weight of 25 lb. a variation of contact pressure of 33 lb., which the rod is amply able to resist. The suspenders are never in compression, and the contact rod need not be supported on springs; a simple hinged connection answers every purpose; and a thin tension rod with eyes, or even a wire, may be used for a suspender. The current collector may be a heavy, strong

and durable roller such as has been shown to be satisfactory by past experience.

A contact wire suspendeded from ropes with 300-ft. spans requires two ropes in inclined planes to prevent excessive lateral motion caused by the wind. The suspenders must be stiff triangular frames. The contact wire must be connected to the suspenders by springs arranged so that they will always operate properly. The wire must be under tension to enable it to resist the pressure of the sliding bow. This tension will vary greatly with changes of temperature. The variation in the temperature of a wire exposed to the sun and carrying a variable and considerable current is, in the Northern United States, 140° fahr., or more. It is not possible to adjust the wire so that it will be satisfactory at both extremes of temperature. It will either stretch and occasionally break at the lowest temperature or be so slack in summer that the collector will jump and cause sparking and excessive strains. If the springs which are necessary at the suspenders do not operate properly, or as intended, the oscillating motion of the collector will be jerky and will cause alternately excessive and defective contact pressure, endangering the wire and causing sparking. The breaking of the wire may be accompanied by the rise of the sliding bow above it, and the destruction of a number of suspenders, causing the fall of the wire and possibly fatal disaster. The currrent collector cannot be a substantial roller; it must be an elastic sliding bow, so as to reduce the oscillating weight to the smallest possible amount. It will be difficult to design a tolerably satisfactory sliding bow, and it will cause at best a large cost of maintenance. The only advantage of the wire is its larger conductivity for alternating currents and the consequent saving in the cost of feeders. In short, a contact wire and its accessories make a complicated, short-lived, and unsafe structure, expensive to maintain, while a contact rod is simple, safe and durable.

The writer has applied for a patent on the contact rod shown.

DISCUSSION.

Mr. Sprague.

FRANK J. SPRAGUE, M. AM. SOC. C. E.—The speaker finds it difficult to discuss this paper as fully as he would like. He will not take up the mechanical theory of construction, so ably worked out, because, as a whole, the paper is a plea for the electric operation of a trunk-line railway by a specific method, on the score of economy, and is based upon the assumption that the various claims made during the past six years for single-phase operation have been justified by accomplished results, which, up to the present, is not a fact.

The arguments advanced seem to be somewhat vitiated by a number of statements concerning both direct-current and single-phase operation which will not bear critical analysis. The speaker, therefore, will make only some random comments.

First of all, he quite agrees with Mr. Mayer in his general conclusion as to the necessity of higher potentials, but is quite at variance with that conclusion if the contention be that these can be effective only through the agency of single-phase alternating currents.

Early High-Tension Proposals.—In a paper by the speaker before the Kansas City Electric Light Convention, in February, 1890, he made the following statement:

"As regards the potential, other things being satisfactory, whatever pressure is demanded in the interest of economical and effective service will be used; and means will be found, consisting mainly in care of construction, which will make its use for the purposes, and, as intended, safe and proper. We have, in these matters, to face the same questions that we have in the matter of steam pressure or of railway speed. To accomplish the larger engineering feats necessary to meet the demands of economy and commerce, we will be governed more by belief in our power to fully subordinate a good servant to our will than by our fears of its vagaries when allowed to become a master."

At the same time, to illustrate the possibilities of electric railway operation, the potential necessary to handle the then existing passenger service on the Philadelphia Division of the Pennsylvania Railroad was determined, the service being rearranged so as to permit of the dispatch of cars every 10 min. The supply was to be by direct current from a single overhead copper rod, 1 in. in diameter, over each track, with return through the rails. Among the solutions, it was found that with two stations 45 miles apart only about 1 800 volts motor potential was required.

It is a curious coincidence that now, sixteen years later, the proposal to use an overhead rod should be resuscitated, and also that the speaker's early proposal of a million circular-mil section

of copper should be about the equivalent of the capacity of the special third-rail now in use. Mr. Sprague.

Mr. Mayer's proposal, however, is for a sectionalized soft steel overhead conductor, 1¼ in. in diameter, and of absolutely the worst known form for the transmission of alternating currents, for it offers the minimum area for unit cross-section, an impedance fully nine times the resistance of the same rod to direct current, and probably eighty to ninety times that of a copper rod of the same size to direct current.

Since the Kansas Convention, the speaker has often advocated the necessity of higher potentials, and is glad to say that, not only in alternating-current work, but in direct-current application, his early prophesies, repeated a year ago with an expression of readiness to undertake the responsibility of direct-current installation at 1 500 volts, even on a third-rail, have, by virtue of the developments of recent months, now been brought within the reach of practical accomplishment. This, under ordinary trunk-line conditions, would mean an increase of fully five times the distance between sub-stations common with 600 volts, and often even the possibility of their entire elimination.

Working Conductors.—Many difficulties would disappear were we in a happy age of wireless power transmission. We deal, however, not with dreams, but practical realities, and among these is the physical means of getting electric energy from a central station to the moving trains. The choice is limited broadly to two general methods, the overhead system in some form, available for both alternating and direct currents, and the third rail, available ordinarily only for direct currents.

Mr. Mayer, after pointing out the shortcomings of present forms of trolley construction, if used for high-speed trunk-line operation, somewhat naively remarks that:

"As a very high and dangerous voltage must be used in the suspended conductor, it is essential to suspend it in such a manner that it will never come down."

The speaker fears that we have not arrived at that millennium in railway operation when collisions, with rearing equipment, and derailments, with consequent destruction of upright structures, are no longer among the possibilities; nor are we beyond the reach of sleet formation, especially on rigid structures, or the ravages of storms of various kinds.

Of course, quite aside from the matter of expense, all conductors necessary for bringing current to a train are objectionable in many particulars, and the points with regard to each were recently well stated by W. J. Wilgus, M. Am. Soc. C. E., at a meeting of the New York Railroad Club, about as follows:

Mr. Sprague.

The third-rail offers certain hindrance to the ordinary maintenance of track, but overhead construction is inelastic, and the laying of additional tracks, or changes in grade and alignment require radical and expensive alterations in permanent overhead structures.

Derailments will crush one form of conductor to the ground, but may equally well knock down the supporting structures of the other and put all tracks out of service.

In wrecking, the third-rail offers some obstruction to the throwing of the equipment to one side, but conductors overhead may interfere with the operation of crane booms.

With two or more tracks, snow cannot well be piled up between them when the space is occupied by third-rails, but, on the other hand, overhead conductors are a source of danger to trainmen, and are subject to the troubles of sleet formation.

There has been much loose talk about third-rail troubles with snow and sleet, complications at frogs and switches, difficulties of current collection, and the danger to employees and trespassers. Extended operation has proven that many of these objections are largely overcome by a protected third-rail, properly designed with due regard to clearance lines of equipment. On the other hand, with an overhead, 11 000-volt trolley, there is certainly danger to the public at overhead street and highway bridges, in tunnels and at low bridges because of the possibility of rearing equipment in case of derailment or collision, and because of the frequent physical necessity of the trolley being brought within a short distance of the cars.

There is, also, corrosion due to locomotive gases when steam and electric operation are maintained over the same tracks, and increased danger to men engaged in cleaning, painting, or repairing overhead structures in close proximity to high-potential lines.

Again, and this seems to be important on high-speed service, there is the possible interception of the train operator's view of the signals, because dips in the railroad grade may bring overhead bridges in front of the semaphores, which likewise may be made less distinctive when having truss members for a background. Nor can we ignore an increased element of danger to the men who have to maintain signal structures, and then there are the objections to exposed high-tension conductors certain to be raised by city authorities.

References to Contracts.—References to contracts where single-phase operation has been adopted are not always conclusive, and the inferences drawn may be misleading unless the circumstances are fully known. Some of those mentioned in this paper come in this class. The Sarnia Tunnel offered an opportunity for a manufacturing company to make a demonstration of the possibilities of oper-

ation by single-phase locomotives. The speaker is quite sure it will not be claimed, by engineer or manufacturer, that this equipment was not well within the possibilities of direct-current application, and it is certainly quite conceivable that the opportunity for first demonstration on a considerable scale had a financial value sufficiently tempting to the railroad company. As an offset to this, it can be stated that, on a similar problem, one set of absolutely comparative bids had demonstrated, for this particular class of work, a marked disadvantage for the single-phase equipment, both as to first cost and cost of operation. Mr. Sprague.

On the London, Brighton and South Coast Railway it is true that a short section is to be equipped with quite another type of single-phase motor, but the reasons which have been publicly advanced, in so far as they are based on motor performance, are debatable, and if the speaker's information is reliable, the cost of installation will be a sorry criterion.

The New York, New Haven and Hartford Railroad project represents a different class of installation: the replacing of steam passenger locomotives by single-phase alternating-current locomotives supplied from an 11 000-volt trolley line. The adoption of this equipment, under the particular conditions which exist, especially the use of a common terminal with the New York Central Railroad, which is to use another type of equipment, has been the subject of a good deal of discussion and some acrimonious comments, many of which have had nothing to do with the system *per se*.

The locomotives on both these roads are of the gearless type, and within a few months there will be opportunity for making comparisons of their performance—the New Haven representing the very latest single-phase developments, and the Central the embodiment of direct-current plans formulated nearly three years ago, and under the then existing limitations of 600-volt operation instead of the much higher potential now practicable.

Among other comparisons will be that of the physical construction and cost of the working conductors, the former using a protected under-contact third-rail, and the latter an overhead double-catenary trolley carried on steel bridges spanning the tracks. Each is interesting in that each represents construction of that type which must be adopted to come within the classification of permanent, that is, of character equal to other parts of the road. A comparison is in order, because the construction proposed by Mr. Mayer will be at least as costly as that adopted on the New Haven road. It is an important fact that the unit cost per mile of track for the overhead construction is fully equal to, if it does not exceed, that for the third-rail, despite the very high potential and the small size of the working conductor. It is proper to add that, no matter

Mr. Sprague. how high this potential, the unit cost of the conductor system alone, per mile of track, for this class of construction will probably not be reduced.

Another feature of the New York Central equipment cannot be compared to a like equipment on the New Haven road, because it does not exist on the latter, and that is the motor-car service, considered by many railway engineers and transportation officials, and applauded by Mr. Mayer, as one of the indisputably good reasons for adopting electricity for suburban service.

The former road is also equipped with batteries, largely because of insurance, and not as an actual necessity of operation. These could not be applied to a single-phase equipment without the introduction of motor-generator sets of the full capacity of the battery discharge, as an essential of supply from the battery to the line, which, in case of breakdown, would eliminate the battery from use; whereas, in the direct-current system, the intermediate machinery—that is, the booster—is simply for regulation, and is of only differential capacity, hence a fraction of the size of the others, and it is not essential to the supply of the current to the line in case of an emergency.

However, these two great systems will, of course, tell their own story, and after a reasonable length of service be influential in moulding decisions as to the future; but it is safe to say that, however ardently the most enthusiastic engineer may preach the beauties of economy, no great trunk systems, except for special reasons, as for example the urgent necessity of increasing capacity, will adopt electricity to save money until after the tangible results have been measured on these two roads. In fact, the probabilities are that, in any case, it will, to quote an official of an English road, be a case of spending money, not to save it because of economy in power, but to make it.

Reasons for Adopting Electricity.—There are but two broad grounds on which the adoption of electricity for trunk-line operation can be justified: First, because of such demonstrated economy, due to the generation of power at large stations and reduction in the cost of locomotive upkeep, as will more than offset the interest and depreciation of that which is essential to electric operation; and, second, the possibility of an increased service of a character which cannot be gotten in steam operation—such, for example, as the handling of trains at increased speeds, or moving heavier trains at satisfactory speeds because of a greater concentration of power, or the operation of frequent motor-car trains at high schedules in suburban districts and between important terminals.

Trunk-line service, however, presents peculiarities of its own, and its demands are responsible for a great awakening in the

matter of higher potentials, which has led to many over-enthusiastic statements, and much discussion as to the merits of single-phase, polyphase, and direct-current equipments. Mr. Sprague.

One thing is certain: higher potentials are necessary and possible. Another is that both motor-car combinations and locomotive-drawn trains must be used, the former necessarily, and the latter preferably under "multiple-unit" control.

Capacity of equipment is essential, and that more than economy. How is it to be gotten? Naturally, first by increase of individual equipment, and then, when this increase has reached its limit, by combining equipments; therefore the question of capacity will be briefly discussed.

Limitations of Design.—Neglecting for the moment the motor car, consider the simplest form of ordinary train operation: What is it that is to be replaced? Simply a steam engine; in other words, a motor supplied by a local boiler, furnace and coal bin, by another motor supplied through a wire by a bigger boiler, bigger furnace and bigger coal bin. It is not sufficient simply that these latter can be made of any desired size; in any case, they are intended to supply a number of motors. That which is essential, and in the last analysis vital, is that the new motor shall not only have certain mechanical advantages, to the extent of eliminating the evils of reciprocating parts, but it must, above all, have capacity, measured not alone by torque or speed, but by both, and it must be of sustained character. Everything else is of tributary character.

The designer of railway apparatus, whether for motor cars or locomotives, is handicapped by certain physical limitations which it is not within his power to change. These are determined by the gauge of the track, the size and number of drivers, the clearance under the motors, the length of rigid wheel base, the dead weight per wheel and the total weight per axle, to say nothing of such a trifling question as cost of construction; and it is his problem to provide within those limitations all the capacity possible.

It is just in this particular that the single-phase motor, despite an amount of experimental work, study and effort of the most persistent and able character, is sadly defective; and, as far as any present knowledge goes, its defects, while not prohibitive to the extent of making it an unworkable machine, are so inherent as to make it practically certain that, when compared with other kinds of electric motors, it is placed, individually, at a very serious disadvantage.

Comparison of Motors.—Of the three types of motors in use to-day in railway work, two—the direct-current and the three-phase motors—have a continuous rate of energy input, while the single-phase has an intermittent and variable rate. In the latter the

Mr. Sprague. attempt has been made to attain the results achieved by the variable-speed direct-current series commutator motor, despite the fact that there is combined in a single machine a motor and a transformer. Now, whatever the hopes of the engineering and railway world, it is necessary to deal with sundry tangible facts.

The first of these facts is, that the single-phase motor, when put to serious duty, and measured by identically the same rating and under like physical conditions, is only about half as good as the direct-current motor. Or, in other words, the weight of the complete electrical equipment, including motors, regulators and car-transforming apparatus, and with like physical limitations, is fully twice that required for direct-current—with, of course, a material increase in the mechanical equipment. It follows that the total weight of a single-phase alternating-current locomotive, to do the same service as a direct-current locomotive, will easily run from 40 to 50 tons more. The reason is simple; it is because of the heat generated on account of lower electrical efficiency, and the impossibility of working the fields of the motors at more than about one-half the magnetic flux.

Polyphase motors have the same advantage as direct-current series motors, in the matter of weight capacity, but are normally constant-speed motors, although it is possible to arrange for half and full speed, each of which is, however, a constant speed.

The speaker has made, and so have others, many independent investigations on this subject, and in the present state of the art the facts stated cannot be gainsaid. Therefore, it is somewhat astonishing to find Mr. Mayer, referring to certain machines, saying:

"These locomotives use high-voltage alternating currents, and have excellent speed regulation, giving practically uniform efficiency at any considerable speed, two advantages which far outbalance, especially for trunk-line traffic, their inferiority in other respects."

This latter is admitted as:

"Great weight per unit of power, limited capacity, high cost, and only moderate efficiency at any speed."

This description of inferiority is an unexpected indictment of the single-phase motor, and, considering a locomotive simply as a machine, he adds to this indictment by his criticism of the motor-generator type of locomotive, where a single-phase motor receiving current from the trolley line drives a generator which in turn supplies direct-current motors, concerning which he says: "Its complications, moderate economy, inevitably high cost, and great weight, are its serious drawbacks," in spite of its "excellent and economical speed regulation." As a matter of fact, the motor of such a combination can be directly supplied from a high-tension line, and the

Mr. Sprague.

weight of this type of equipment is almost identical with that of a single-phase equipment, while it has some possibilities which the latter has not.

A practical illustration of the bearing of this difference in capacity is the fact that although the essential weight on the drivers of the New York Central machines is about 70 tons, instead of 85, as on the New Haven, the individual capacity of the former is so much greater that, with equal ventilation, fifteen of them will probably do the work of twenty-five of the latter.

Lack of capacity, however, is not the only feature wherein the single-phase motor fails in comparison with the direct-current machine. On large sizes, instead of two or four poles it has ten or twelve, and the series winding on the armature with two paths is replaced by one with as many paths as there are poles. The armature, in machines of similar capacity, has a larger diameter, and runs at a higher speed; and, instead of two fixed sets of brushes, there are as many movable sets as there are poles, to supply current to a commutator of larger size. The clearance between field and armature is about one-quarter that of the direct-current machine, and the allowable bearing wear, therefore, is much less. Finally, the New York Central type of machine, with its remarkable mechanical as well as electrical advantages, is impossible to direct alternating-current construction and operation.

It is curious to note with what persistency certain well-sounding statements reappear, in spite of frequent disproof. Among these may be mentioned that referring to possible economic variations of speed. With proper control, the ordinary four-motor locomotive operated on direct current has three impressed electromotive forces, for each of which it has a wide automatic range of torque and speed far beyond the limits, as the sustained torque increases, of the single-phase motor; and for each of these established conditions there is a very considerable range of speed with the same torque, provided the motors are properly constructed and controlled. The result is that, as every one of these speeds is an economic one, all variation essential for practical railroad operation is provided.

Direct-Current Motor Improvements.—Comparing the condition of the art at present with that of about two years ago, it will be seen that three important and radical developments have taken place in direct-current motor construction. The first is the use of the commutating pole, which, in a large measure, and certainly for any ordinary pressures, has eliminated the much abused bugbear of the commutator to such extent that, when properly constructed, it is impossible to tell by ordinary inspection whether a machine is running at high or low potential, or with large or small current, so absolutely free is it from sparking.

Mr. Sprague.

This has led to additional facilities in control, and the construction of motors for operation at from 1 200 to 1 500 volts, with the further possibility of operating two in series at double potential.

A third development is that illustrated by the New York Central gearless locomotives, in which the hitherto invariable practice of maintaining a fixity of relation between the armature, or rotating part, and the field magnet, or fixed part, has been abandoned, the armature being mounted directly on the axle, and the field magnets forming a part of the locomotive frame, supported by its springs and hence movable with regard to the armature. In this construction, therefore, there are no armature or axle bearings. This machine is one of the most remarkable and satisfactory outcomes of recent work, and, considering actual necessary electric additions to the mechanical equipment, it has, when designed for the conditions under which it should operate, probably the highest weight efficiency of any direct-current locomotive. Being of bi-polar construction, with a wide range of neutral brush contact, it runs practically sparklessly at all loads, is a natural 1 200-volt machine without commutating poles, and can easily be wound for a higher potential if using such poles.

In general, considering briefly the first cost of installation, it has been shown that the weight and cost of motor equipment are much higher for the single-phase alternating-current than for the direct-current system, and that the cost of permanent working conductors per mile of track is fully as high. The cost of the central station will be somewhat greater because of the reduced capacity per weight of material of single-phase generating apparatus. There remains the intermediary apparatus, when extended systems are considered, namely, static transformers and rotaries for the direct-current system, and sub-station and car transformers for the alternating-current system.

Unless there is material increase in efficiency and capacity of single-phase motor equipments, it is quite likely that the interest on the increased cost of sub-stations of one kind over that of the other, and the losses in the rotaries will be at least equalled, if not more than offset by the interest on the increased first cost and the cost of operation of the motor equipments on the alternating-current system, when each is installed with equal regard to permanency.

Every engineer, of course, hopes for, and will welcome actual advances in this art, by whatever system—everything, in fact, which makes for increased economy, added reliability and reduced cost of operation—to the end that there may be a greater extension of electric operation; but it is absolutely essential to compare results accomplished, not merely promises of performance.

Such are some of the general facts which must be confronted

in dealing with the question of electric operation of trunk lines. In conclusion, the speaker will quote a remark by Chief Engineer Kando, of the Ganz Company, of Europe, the advocates of polyphase motor operation, in a report recently made to his company after a trip in the United States. Referring to the question of equipment, he said: Mr. Sprague.

"That system will prove to be victorious which can present the most powerful locomotive, and the locomotive which will possess the greatest capacity for a given weight."

The speaker might add to this, and say that it would seem that that type of locomotive must ultimately survive which has not only the greatest weight capacity, but also automatic response in torque and speed to the varying demands of grade and curvature.

JOSEPH MAYER, M. AM. SOC. C. E. (by letter).—The main subject of this paper is a comparison of steam-locomotive and electric operation by a cheap water-power. Incidentally, the different methods of electric operation are briefly mentioned therein in order to justify the choice of a particular method for the comparison. The paper also discusses in detail the contact conductor required. Mr. Mayer.

The writer has endeavored to show that a copper wire, under tension and suspended by stiff frames 10 ft. apart, from two ropes in inclined planes, is mechanically unfit to resist with adequate safety the incident forces due to changes of temperature and the variable pressure of a sliding bow or roller moving at a speed of 80 miles per hour. The main difficulty arises from the variation in the tension of the wire caused by changes of temperature. The minimum tension required in summer, together with its increase by the fall of temperature, is about equal to the elastic limit of a large copper wire; this leaves no margin for the forces due to the rapidly moving sliding bow, and must result either in the stretching and occasional fracture of the wire in winter or in inadequate tension in summer. The writer endeavored to remove the variation in the tension of the contact wire arising from changes of temperature by an economical automatic expansion joint. His efforts in this direction led him to the conclusion that such an expansion joint is not practicable.

One method of avoiding this difficulty is to use a contact conductor able to resist the incident mechanical forces, without being under tension. In this case, the expansion joint becomes a simple problem. A 1-in. copper rod, suggested by Mr. Sprague, would cost in place at least $4 000 per mile of track. A smaller rod would be mechanically inadequate. A soft-steel rod is a poor conductor, for alternating currents, and would have to be reinforced by a feeder frequently cross-connected to the rod. With alternating-current operation at 15 000 volts, the currents are small and the combina-

Mr. Mayer.

tion proposed by the writer gives a moderate drop of voltage with a loss of only 4% of the energy transmitted. The drop of voltage and the amount of self-induction can be reduced still further if desirable, by introducing, at small cost, one additional cross-wire per mile of track from the feeder to the contact rod in series with a closely adjusted fuse.

A feeder must be used with any kind of conductor, if a high degree of reliability is desired. Where the steel rod is in parallel with the feeder, it carries but a very small current; where it is in series with the feeder, it carries, at worst, one-half the current for one train for one-half the distance of the cross-connections to the feeders. The rod must be round to receive the minimum wind pressure, and to permit the use, without excessive lateral deflection, of a single carrying rope of long span for its support. Mr. Sprague considers mainly the electric features of the contact conductor, but the problem is believed to be mainly a mechanical one. The writer's contact conductor with the feeder costs less than one-third as much as the rod proposed by Mr. Sprague. It is only fair to state that his remarks were written within a day, and are probably based on calculations made some years ago when prices and railroad equipments were different. His assertions, the writer has no doubt, are true for the assumptions which he made then, but they are expressed in more general terms than the facts justify, and the qualifying assumptions are not explicitly stated, but may be guessed at by the judicious reader. Mr. Sprague's eminent qualifications, however, will lead many readers not familiar with the subject to the opinion that the writer has made a serious error in selecting for comparison the alternating-current operation, and that his views on the relative merits of the two kinds of operation are unduly favorable to the one selected.

Different kinds of operation are different methods of producing the energy developed at the rim of the drivers. This is used to haul the cars and the motive power equipment. The former is the useful or net energy. The locomotives and motor car equipments are rated according to their total output of energy per second at the rim of the drivers.

In comparing different kinds of equipment, their difference in weight per unit of power (or energy output per second) must be considered, and the power needed to haul this difference must be charged to the heavier equipment. For comparative purposes, a convenient unit of energy is the horse-power-year, which is:

$$365 \times 24 \times 3\,600 \times 550 \text{ ft-lb.}$$

The writer has shown* from the official reports of various railroads, and from the most recent tests of train resistance, that the cost of producing with steam locomotives one horse-power-year at

* *Railroad Gazette*, November 16th and 23d, 1906.

the rim of the drivers is approximately $225, excluding interest charges. Mr. Mayer.

In the following the writer will show the cost of direct-current and alternating-current electric operation per mile of line and per horse-power-year at the rim of the drivers for different amounts and kinds of traffic on single-, double-, and four-track roads. This is the only way to answer Mr. Sprague, and to judge the merits of different methods and the claims of their advocates.

The cases considered are:

I d and I a.—Direct-current and alternating-current operation for a single-track road with an average of 9 trains per day, the largest power consumption for one train being: At full speed, 750; and at starting, 1 000 kw. direct-current. Alternating-current equipments are heavier, and the power consumption per train at the rim of the drivers is taken with them 7% larger. It is assumed that one-quarter of the trains will use each the maximum amount of power; one-half of them each two-thirds, and one-quarter of them each one-third as much power. These assumptions give a plant suitable for average single-track steam-railroad traffic.

II d and II a.—Direct-current and alternating-current operation of a double-track road with 80 trains per day, the power consumption per train being the same as under *I.*

III d, III a and III h v d.—A double-track road with 80 trains per day, the largest power consumption per train being 1 500 and 2 000 kw. direct-current at full speed and starting, and 7% more at the rim of the drivers with alternating current. One-quarter of the trains using each the maximum amount of power, one-half the trains each one-half, and one-quarter each one-quarter as much power. *III h v d* gives the costs for direct-current operation with 3 000 volts in the contact conductor.

IV d and IV a.—Direct-current and alternating-current operation of a four-track road with 240 trains per day of the sizes given under *III.*

The estimates are based on the use of steam-turbine power-stations at the most economical distance. The voltages for direct-current and alternating-current operation are 700 and 15 000.

The final results of the calculation will be given first, and, afterward, their details.

The cost of the installation per horse-power-year developed at the rim of the drivers of direct-current equipments, and per 1.07 h-p-year, with alternating-current equipments is shown in Table 2.

The cost of operation per horse-power-year developed at the rim of the drivers with direct-current operation, and per 1.07 h-p-year with alternating-current operation, together with 5% of the cost of the plant, for interest and taxes, is shown in Table 3.

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TABLE 2.

$I\,d = \$3\,317$	$I\,a = \$1\,065$	.....
$II\,d = 991$	$II\,a = 674$	.....
$III\,d = 852$	$III\,a = 552$	$III\,h\,v\,d = \$480$
$IV\,d = 462$	$IV\,a = 423$	.....

TABLE 3.

$I\,d = \$407$	$I\,a = \$174$	.....
$II\,d = 170$	$II\,a = 139$	.....
$III\,d = 150$	$III\,a = 114$	$III\,h\,v\,d = \$104$
$IV\,d = 106$	$IV\,a = 103$	.....

Steam locomotives with their tenders, especially for passenger trains, are much heavier than electric locomotives and motor-car equipments; therefore, for a given amount of traffic, they must develop more power than the latter; allowing for this advantage and for the interest saved on the proceeds from their sale, the writer believes that $225 is a very low estimate for the cost of the steam-locomotive power required for the traffic moved by one horse-power-year at the rim of the drivers of direct-current equipments. Assuming the same number, size and speed of trains for steam-locomotive and electric operation, then, by the foregoing tables, the net return on the investment required for the change in the motive power may be calculated. It is here assumed that the amount of traffic and the receipts from it remain the same. The percentages of net return for the different types of electric operation are shown in Table 4.

TABLE 4.

$I\,d = -1.5$	$I\,a = +8.8$	.....
$II\,d = +9{\cdot}5$	$II\,a = +16.7$	.....
$III\,d = +12.8$	$III\,a = +25.3$	$III\,h\,v\,d = +29.2$
$IV\,d = +29.7$	$IV\,a = +32.9$	.....

Table 2 shows that the cost of installation is in all cases largest for direct-current operation at 700 volts. The difference is very large for single-track and small traffic; it is much less, but still considerable, for double-track and large traffic, and is small for four tracks with the very large traffic assumed.

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An inspection of Tables 3 and 4 shows the superiority of alternating-current over 700-volt direct-current operation for single- and double- track. For four tracks, the two kinds of operation are nearly equally economical with the traffic assumed.

High-voltage direct-current operation promises to be the most economical method with two or more tracks and large traffic. If these estimates are approximately correct, the writer was fully justified in choosing, for double-track, single-phase alternating-current in preference to direct-current operation at 700 volts. High voltage direct-current operation was not considered, because it is not at present available.

The foregoing estimates will not be accepted without knowledge of the methods by which they were obtained. It is necessary, therefore, to give the details of at least a part of the estimates of which the results are shown in Tables 2, 3 and 4. These will explain the reasons for the large differences in the cost of power production per horse-power-year at the rim of the drivers with alternating-current and direct-current operation of the different amounts and kinds of traffic on single-, double-, and four-track lines.

Direct-Current Operation of a Double-Track Trunk Line (III d of Tables 2, 3, and 4).—The amount of power used by the largest and fastest trains determines the capacity and cost of the transmission plant required; therefore, it is of the utmost importance. This question has undoubtedly been investigated carefully by the electrical commission appointed to study the New York Central electrification at New York. Their conclusions are embodied in thirty large locomotives, with Sprague multiple-unit control, of 2 200 h. p. sustained and 3 000 h. p. hourly capacity. These locomotives, with 90% efficiency, require a current of 1 827 to 2 487 kw. Provision is made for operating two of these in one train. This would require twice as much current. On many trunk lines there is no occasion for such a large amount of power for one train. A current of 1 500 kw. for a train at full speed and one of 2 000 kw. at starting will here be assumed as the maximum. This will take care of heavy, though evidently not of the heaviest, future trunk-line traffic. Taking the Baldwin Locomotive Works formula, $R = 3 + \frac{V}{6}$, for the train resistance per ton of heavy passenger trains, and allowing 30 kw. for lighting the train, we obtain, with 90% efficiency of the direct-current locomotive, at full speed, for a maximum speed of 70 miles per hour, on a level tangent, a permissible train weight of 650 tons including the locomotive. Under favorable conditions much heavier trains can be hauled at this maximum speed. Schedule speeds of 55 miles per hour for express trains of the above weight become thereby practicable. Under un-

Mr. Mayer. favorable conditions, causing a train resistance of 7 lb. per ton, it will be possible to haul a 3 000-ton freight train at a speed of 30 miles per hour, on a level tangent.

The number and size of trains given in Tables 2, 3 and 4 under *III* will be assumed. The average power consumption of each train will be assumed to be equal to full-speed power, the excess at starting being balanced by the deficit during stops. The maximum grade is assumed to be $\frac{1}{2}\%$ or less. The direct-current operation requires a low-tension circuit consisting of two third-rails, four bonded track rails, and feeders; rotary converter sub-stations, high-tension lines, and power-stations.

The Low-Tension Circuit for the Direct-Current Operation of a Double-Track Line.—When a train moves from the sub-station to the center of a section, the voltage received by the motors drops from the sub-station voltage of 700 to a much smaller amount. This drop is largest when two trains using the maximum amount of power pass each other at the center of a section, one of them at full speed, the other under acceleration. The case of two maximum trains in such position starting at the same time is too rare and unnecessary an occurrence to require consideration.

A drop of 175 volts in the low-tension circuit under the above exceptional circumstances is permissible. A variation of voltage between 700 and 525, however, is so large that the assumption of constant voltage at the motors would lead to very erroneous results in regard to the amount of current to be supplied at different distances from the sub-stations.

Allowing for the variations in the current due to differences in the voltage at the train, calculation shows that the average drop when one maximum train passes alone over a section between two succeeding sub-stations is 61 volts. This drop is increased to an average of 70 volts by the trains on the other track. This gives 630 volts as the average pressure of the current supply at a maximum train. The average current supplied to such a train, therefore, is $\frac{1\,500}{0.630} = 2\,381$ amperes. This train will receive, at average speed and 700 volts, that is, near a sub-station, $2\,381 \times \frac{00}{630} = 2\,646$ amperes. It will receive, at average speed near the center of a section, at 525 volts, $2\,381 \times \frac{525}{630} = 1\,984$ amperes. Two trains, one starting, will require $1\,984 \times 2\frac{1}{3} = 4\,629$ amperes. The low-tension conductors consist of two bonded tracks cross-connected at frequent intervals and two third-rails also cross-connected. One-half of the current comes from each direction. The current in one third-rail and in one track, therefore, is $\frac{4\,629}{4} = 1\,157$ amperes.

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This current must be sent through 3 miles of third-rail and 3 miles of track. The proper combined resistance of the forward and return circuit, therefore, is $\frac{175}{1\,157} = 0.1513$ ohms. Economy requires equal resistance for the forward and return conductors, or 0.0756 ohms for each. With two 90-lb. track rails, the area of the return conductor is 17.64 sq. in., equal to $17.64 \times 1\,273\,000 = 22\,460\,000$ cir. mils. The equivalent copper area is $\frac{22\,460\,000}{13} = 1\,727\,700$ cir. mils. The resistance of 3 miles of this conductor is $\frac{11 \times 5\,280 \times 3}{1\,727\,700} = 0.1009$ omhs. The conductivity is $\frac{1}{0.1009} = 9.911$ mhos. The conductivity required is $\frac{1}{0.0756} = 13.228$ mhos. The conductivity of the feeder, therefore, must be $13.228 - 9.911 = 3.317$ mhos. The circular mils of the return feeder must be $3.317 \times 11 \times 5\,280 \times 3 = 577\,100$.

The conductivity of a 90-lb. per yard bonded third-rail 3 miles long is 7.16 mhos. The conductivity of its feeder must be $13.228 - 7.16 = 6.068$ mhos. The area of the feeder is $6.068 \times 15\,840 \times 11 = 1\,057\,200$ cir. mils. The total copper area for the feeders for double track is $2\ (577\,100 \times 1\,057\,200) = 3\,268\,600$ cir. mils. The weight per mile is $3\,268.6 \times 5.28 \times 3.02 = 52\,120$ lb. Allowing for cross-connections, say 53 000 lb., these cost, at 22 cents per lb. in place, $11 660 per mile of line.

It remains to determine the efficiency of the low-tension circuit. The average drop was found to be 61 volts when one maximum train passes over a section. The average power consumption per train is only nine-sixteenths of that of a maximum train. The average drop, when an average train goes alone over a section, therefore, is $\frac{9}{16} \times 61 = 34.3$ volts. This is increased by trains passing at the same time on the other track. When there is an average train passing in the opposite direction on the second track, while an average train passes over a section. The average power consumption per train 48 volts. When there is one and one-fourth times the average traffic, there is a train on a section of one track during $1\frac{1}{4} \times \frac{1}{3} = \frac{5}{12}$ of the time. There is a train on each of the two tracks of a section during $\frac{5}{12} \times \frac{5}{12} = \frac{25}{144}$ of the time. There is a train on the first track and none on the second track during $\frac{5}{12} \times \frac{7}{12} = \frac{35}{144}$ of the time. There is a train on the second track and none on the first track during $\frac{35}{144}$ of the time. With one and one-

Mr. Mayer. fourth times the average traffic, there is, therefore, a train on only one track of a section during $\frac{70}{144}$ of the time.

When there are trains on the two tracks of a section, the average drop is 48 volts. When there is a train on one track, the average drop is 34.3 volts. Two trains take about twice the current of one train, the amounts of power to which the two drops apply, therefore, are, as 50 to 70, and the average drop is $\frac{50 \times 48 + 70 \times 34.3}{120}$ = 40 volts. This gives a loss of 5.7% in the low-tension conductors when the traffic is one and one-fourth times the average amount. This is also approximately the average loss of power in these conductors. The loss of power in a conductor increases as the square of the current; the average loss of power with a variable current, therefore, does not occur with the average, but with a larger current. This average loss can only be calculated accurately if the current is known as a function of the time. The calculation here given is only approximate. It is given in detail because it forms the basis of the whole estimate of the cost of direct-current operation.

For the calculation of the capacity of the power-station, one and one-half times the average amount of traffic would have to be taken. This traffic, however, will be distributed unequally over the different sections. The average loss in the low-tension conductors is found by taking about one and three-fourths times the average traffic. Therefore, it is $5.7\% \times \frac{7}{5} = 8$ per cent. In the calculation of the feeders for the low-tension circuit, the assumption was made that the train speed is an average when two maximum trains meet at the center of a section. In reality, the train speed, in this case, is less than an average, since the presence of another maximum train in the same section reduces the applied voltage and the current supplied to the motors. The result of this assumption, therefore, is that, on account of the smaller train speed than was assumed in the calculation, the current, when two maximum trains meet at the center of a section, is larger than calculated, and the drop is more than 175 volts. The amount of feeder copper would have to be somewhat larger to keep the maximum drop below this limit. The maximum of 175 and the average of 40 volts are smaller than the most economical drop in the low-tension conductors. Larger drops would mean larger sub-stations and greater speed variations of the fast trains. For these reasons, they are not considered desirable, though they would reduce slightly the combined cost of operation and interest charges.

The Rotary Converter Sub-Stations for a Double-Track Line.—It was found that the current used by one maximum train at full

speed, when passing a sub-station, is approximately 2 646 amperes. Mr. Mayer.
Two maximum trains, one starting, require $2\,646 \times 2\frac{1}{3} = 6\,174$ amperes at 700 volts, or 4 322 kw. This is approximately the maximum load on a sub-station. Several modifying causes were neglected in this calculation. First, the drop of voltage in the transformers and the rotary converters in the sub-station is much increased by the maximum load, and the voltage supplied at this time is less than 700; this causes a smaller current consumption by the locomotives, and a flow of current from the two neighboring stations. There may be other trains on one or both of the adjoining sections, however; in this case the sub-station here considered may have to supply a part of their current. Considering all the factors, the maximum sub-station load may be reasonably estimated at 4 400 kw. If an allowance of 100% is made for overload for a few minutes, a rated capacity of 2 200 kw. is needed, and if 25% reserve is provided 2 750 kw. will be required. The cost of a sub-station of this size is about $34 per kw., or $93 500. This amounts to $15 583 per mile of line. Storage batteries, if used, would decrease greatly the required capacity of the transformers and rotary converters, and would improve their average efficiency, but the efficiency of the batteries is only about 75%; the number of kilowatt-hours consumed, therefore, would be increased, and the load factor and the reliability of the service would be improved. The total costs, with steam power, would be decreased but little if any.

The average load on a sub-station is the power required by two-thirds of an average train increased by the average loss in the low-tension circuit, or

$$\frac{2}{3} \times \frac{9}{16} \times 1\,500 \over 0.943 = 597 \text{ kw.}$$

The load factor, therefore, is $\frac{597}{4\,400} = 0.1357$. The rated capacity of the station is 2 750 kw. The average load, therefore, is $\frac{597}{2\,750} = 0.217$ of the rated capacity. The changes in the load are so rapid that it is impossible to change the number of units in service with the amount of load. The transformers and the rotary converters carry inevitably, unless storage batteries are used, most of the time only a small fraction of their rated full load. The average efficiency of the station, therefore, will be much less than that of a sub-station for an elevated road or a street-car system, where the load factor is often four times as large. The writer estimates the average efficiency of the converters, under these unfavorable conditions, at 87%, and that of the transformers at 95%; the station efficiency is then $0.87 \times 0.95 = 82.6$ per cent.

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Cost of the Protected Third-Rails.—Taking the cost of the special third-rails to be 2 cents per lb. delivered, the cost of a 90-lb. rail for one mile becomes $3 168. Estimating the cost of placing, welding, and of the bonds, the insulators and the wooden protection at $1 932 per mile, gives $5 100 as the cost per mile of a protected third-rail.

The Towers and the Copper for the High-Tension Line.—The cost of a line of steel towers and insulators for carrying the high-tension line and the feeders is $1 600.

With the power-stations 60 miles apart, with three-phase transmission at 15 000 volts, without step-up transformers, with an average power factor of 95%, the copper required for 3% average loss in the transmission costs in place, at 22 cents per lb., $3 400 per mile of line.

Total Cost and Efficiency of Plant.—The cost per mile of the plant for the direct-current operation of a double-track line, therefore, is as follows:

Two 90-lb. protected third-rails, circuit breakers, fuses, switches, and their housing	$11 200
Track bonds	1 000
One row of steel towers for high-tension line and feeders	1 600
Copper for feeders	11 660
Sub-stations	15 583
Copper for high-tension lines	3 400
Total	$44 443

This does not include the power-stations and the electric locomotives and motor-car equipments. Taking the average efficiency of the locomotives and motor-cars at 80%, the total average efficiency from the driving wheels to the power-station becomes: $80 \times 0.943 \times 0.826 \times 0.97 = 60.5$ per cent.

The Alternating-Current Operation of a Double-Track Line (III a of Tables 2, 3 and 4).—With the alternating-current operation, no rheostats are required. A much more efficient start is thereby secured. This increases the average efficiency, especially with frequent stops. For trunk-line traffic, this average efficiency of the gearless locomotives, and the motor-cars, or the combined efficiency of the motors and transformers, is here estimated at 75 per cent. Alternating-current equipments for given weights and speeds of trains, therefore, require more power than those for direct-current in the ratio of $\frac{0.80}{0.75} = 1.067$.

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The alternating-current locomotives and motor-car equipments are much heavier than those for direct currents. The resulting increase of the train weights is estimated to amount to an average of 7 per cent. The power needed is thereby increased in about the same ratio. Both these causes combined increase the power to be supplied to the trains in the ratio of $1.067 \times 1.07 = 1.142$. The trains of maximum power consumption, therefore, require $1\,500 \times 1.142 = 1\,713$ kw. for full speed, and al arger amount during acceleration. The increase of current during acceleration is here estimated at 25 per cent. This makes the maximum power, on a level, $1.142 = 1\,713$ kw. for full speed, and a larger amount during accelas much power. With one and one-fourth times the average amount of traffic, the trains are an average of 7.2 miles apart. The average amount of power used per mile of line is then $\frac{1\,713}{7.2} \times \frac{9}{16} = 133.9$ kw. When the power is supplied at an average voltage of 15 000 with an average power factor of 85%, the current required per mile of line is $\frac{133.9}{15 \times 0.85} = 10.5$ amperes. If the power-stations are 60 miles apart, $10.5 \times 30 = 315$ amperes must be sent out from the power-station in each direction. The power to be delivered to 30 miles of line is $133.9 \times 30 = 4\,017$ kw. To distribute this power uniformly over a distance of 30 miles with conductors of uniform section throughout their length requires one-third as much copper as to transmit the whole amount a distance of 30 miles with the same percentage of loss.

If two 0000 feeders are used, with frequent cross-connection to the contact rods, the latter carry over most of their length but little current. The efficiency of the contact line is approximately the same as when the two 0000 wires are used as contact conductors without other feeders. It is 97% with one and one-fourth times the average load. The cost of the power-transmission circuit, inclusive of the towers, contact rods, wire ropes, steel bridges, insulators, feeders and track bonds, is $11 200 per mile of double track. This includes a liberal allowance for the tower foundations, which vary considerably with the nature of the line.

The total consists of the following items per mile of line:

Supporting steel structure, insulators, circuit breakers, lightning protection and foundations	$6 030
Contact rods and suspenders, with expansion joints and bonds	1 800
Ropes or messenger cables	1 420
Two 0000 feeders	1 600
Track bonds	350
Total	$11 200

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Steel is estimated at 3½ cents, ropes at 8 cents, and copper at 22 cents per lb. erected. Concrete is estimated at $8 per yd. in place.

The efficiency from the drivers to the power-station is $0.75 \times 0.97 = 72.75$ per cent. For a fair comparison with the direct-current operation, the fact must be considered that the loads will be 7% larger, due to the heavier locomotives and motor-car equipments. This is equivalent to an efficiency of $\frac{72.75}{1.07} = 68\%$, and the same loads as with direct-current operation. For the latter an efficiency of 60.5% was found, or 89% as great as with alternating currents. The power-stations for direct-current operation must furnish 11% more power than those for alternating-current operation. The former, however, contain three-phase, the latter single-phase generators. The single-phase generators are more expensive, for the same capacity, and less efficient. The difference in the cost of power for the two kinds of operation, therefore, is small.

When the power is supplied from a distant water-power station alternating-current operation requires the introduction of step-down transformers located along the line, and the high-tension transmission must be single-phase, or special step-down transformers must be used for changing the three-phase high-tension current into two-phase current of lower voltage. The two phases can then be used on alternate sections of the line. The transformer stations can be placed nearer together than the steam-power stations previously assumed, and the area of the feeders can be reduced. The contact rods may be made in short, insulated sections, fed at their centers, so that they carry the current for only short distances. The increase in self-induction caused by their use is then reduced to a very small amount. Most of the self-induction of the transmission circuit is in every case due to the use of the rails for carrying the whole return current over the whole length of the transmission line. With direct-current operation, the average power factor of the load of the transmission line is here estimated at 95%, and it may be even greater. With alternating-current operation, this power factor is estimated at 85 per cent. This increases the amount of copper required for the long-distance transmission in this latter case. With a distant water-power the difference in the cost of power at the exit from the step-down transformer station and at the entrance to the rotary-converter stations is practically nothing.

Therefore, with the direct-current operation, the cost of the plant per mile of line is $44 443, and that of the alternating-current operation is $11 200. With the former there are sub-stations every 6 miles which require continuous attendance. This attendance

costs about $4 000 per year per station, or $667 per year per mile of line. With alternating-current operation there is nothing to balance this expense. There remains the comparison of the moving-power equipment, and the total costs of the two kinds of operation. Mr. Mayer.

Comparison of the Moving-Power Equipment, and the Total Costs of the Two Kinds of Operation.—The average number of trains per day is 80, and per year $365 \times 80 = 29\,200$. There are, therefore, 29 200 yearly train-miles per mile of line. The average power required on a level for an average train at full speed is 844 h.p. Where there is a 0.5% grade the draw-bar pull for a freight train is about two and one-fourth times as great as on a level. On a very long grade, this means the use of an additional locomotive, and, with shorter grades, the use of more powerful locomotives for all trains. The through freight trains going one way are generally much heavier than those in the opposite direction. The ruling up-grades and the weight of the trains in the same direction, therefore, determine the average capacity of the freight locomotives required, and this is much greater than that for an average train on a level. The required capacity of the passenger locomotives is not affected much by infrequent grades of less than 0.5 per cent. The speed of these trains may be reduced one-half by series parallel control, or, with alternating current, by an equivalent change of the voltage. This reduction of speed reduces the train resistance nearly as much as the grade increases it. The motor current, therefore, may remain nearly the same as on a level, and no increase in the capacity of the motors is required. For suburban passenger traffic, with frequent stops and high speeds, where rapid acceleration is essential for good service, the capacity of the motors is several times the average power consumption. This tends to increase the cost of the moving-power equipment, but the frequent trains with this traffic diminish the delays at the terminals and increase greatly the mileage run per year; this reduces the cost per mile of the motor-power equipment. It is evident that the required capacity and the cost of the locomotives and motor-car equipments depend on a large number of widely varying factors, all of which must be considered in an exact estimate. The exact number of miles run per year by the electric locomotives and motor-car equipments must be ascertained from future experience. At present, approximate inferences from experience with the motor cars of interurban electric roads are the principal guides in forming an opinion. The experience with street cars, allowing for the difference in the average speed, and for their smaller delays at the terminals, gives also some assistance in forming a correct opinion. Approximate estimates can also be based on the experience with steam locomotives, allowing for the absence of delays due to coaling, firing up, and the taking of water,

Mr. Mayer. and for less frequent and quicker repairs with electric locomotives, also for the greater speeds which will be adopted with the latter, and for the probably much larger number of trains. Disregarding for the present the switching service, a minimum of 73 000 miles per year per electric locomotive or motor car, with the number of daily trains assumed herein, is a very conservative estimate. This mileage per locomotive or motor car per year, as will appear in the following, is of the utmost importance in judging the relative costs of the two kinds of operation here compared. It will vary with the speeds adopted and also with the number of daily trains, as a large number of trains will reduce the delay at the terminals. A judicious weighing of all the known experience would permit even now a closer estimate of the probable mileage with different numbers of daily trains. To cover all the trunk lines in this investigation, the most convenient course is to estimate this mileage at its minimum value under the most favorable trunk-line conditions, and to remember that all the comparisons are based on this hypothetical value. They can be modified easily as soon as the actual mileage becomes known from future experience with trunk-line traffic, or from the more detailed investigation of past experience. With the above mileage there will be needed one train power equipment for every 2½ miles of line. Estimating the average cost of such a power equipment at $25 000 for the direct-current operation, gives $10 000 per mile as the cost of the direct-current locomotives and motor-car equipments. The corresponding cost with alternating-current operation may at present be estimated at $17 500 per mile of line. Therefore, the total costs per mile, exclusive of the power-stations, are, for direct-current operation, $54 443; for alternating-current operation, $28 700. Taking interest, taxes, cost of maintenance and depreciation in both cases at 8% for fixed plant and 14% for moving machinery, the yearly cost of these items is $5 315 per mile of line for direct-current, and $3 346 for alternating-current operation. To the former the yearly cost of the sub-station attendance, amounting to $667 per mile of line, must be added, as this cost is absent with alternating-current operation. This makes the relative saving by alternating-current operation $2 636 per mile of line per year. To this must be added the saving of about $610 per year in the cost of steam power. The total yearly saving by alternating-current instead of direct-current operation by steam power, therefore, is $3 246 per mile of line.

The assumptions made in this investigation are favorable throughout to direct-current operation. Trunk-line traffic calls for the large trains and high speeds assumed. A greater occasional crowding of trains than assumed in these estimates is now of frequent occurrence. Special rules would have to be framed and ob-

served to prevent it if direct-current operation, with the capacity of sub-stations here provided, were adopted. With alternating-current operation of the kind described, local crowding of the trains would cause no trouble whatever, and much larger and fewer units of the same total capacity might be used, without change in the plant or increase in the cost of operation. The number of trains assumed cannot be increased materially on a double-track line with fast and slow passenger and freight trains. It is evident that a large mileage of passing and side tracks would be required to operate even the number of trains assumed. To avoid excessive complication, these additional tracks have been omitted from the estimates. Evidently their consideration would increase the difference in the yearly cost of the items considered. A diminution in the number of trains would increase the relative advantage of alternating-current as compared with direct-current operation. It is evident, therefore, that with the assumed, and still more with lower, voltage, direct-current operation is uneconomical for the operation of the whole traffic of double-track trunk lines. Mr. Mayer.

Direct-Current Operation of a Four-Track Line (IV d of Tables 2, 3 and 4).—With four tracks, the slow and the fast trains may be separated, and a much larger number of trains per track thereby becomes practicable. The eight cross-connected track rails form a return circuit of low resistance, and the four cross-connected third-rails form a low-resistance conductor for the outgoing current. A great saving in the feeders is evident, and the sub-stations can be placed farther apart. The large number of trains increases the difference in the total cost of the direct-current and the alternating-current locomotives and motor-car equipments. All these changes will reduce greatly, if not remove entirely, the disadvantages of the direct-current operation. A special investigation, therefore, is required for deciding which is the more economical.

Assuming 240 trains per day, three times as many of each kind as for double track, gives an average of one train for every 3 miles. This train, with direct-current locomotives, requires an average of 844 kw. With so many trains the dimensions of the low-tension conductors are governed by the economical average loss of power and not by the greatest permissible maximum drop of voltage. The average loss of power occurs with about one and one-fourth times the average number of trains. With the sub-stations 7½ miles apart, and with one 0000 copper feeder for the four third-rails, the average loss of power in the low-tension conductors is 3.7 per cent.

The required capacity of the sub-stations, in this case, is governed by the condition that their least available capacity should not be exceeded by their average load for several hours in succession. This condition will be fulfilled if the average load with

Mr. Mayer. one and five-eighths times the average number of trains does not exceed the capacity always available. The average load per sub-station is then $2\frac{1}{2}$ trains $\times$ $1\frac{5}{8}$ = $4\frac{1}{16}$ of an average train. The average train takes from the sub-station 844 kw. $\times$ 1.048 = 885 kw. The corresponding sub-station load for $4\frac{1}{16}$ trains is 885 $\times$ $4\frac{1}{16}$ = 3 595 kw. If the station is given a capacity of 4 500 kw., there is 25.1% reserve. The sub-station capacity, inclusive of the reserve, should be kept in operation whenever available, and disabled machines should be replaced quickly; if this is done, overloads will be extremely rare. The cost of these sub-stations is about $16 800 per mile of line. The load factor, and therefore the average efficiency of these stations, is high. That of the rotary converters may be estimated at 92, that of the transformers at 97; the total efficiency, therefore, may be estimated at 89.2. Taking the heat-power stations along the line 45 miles apart, and assuming a 3% loss in the high-tension lines, gives, with 15 000 volts and 95% power factor, $5 200 as the cost of the copper per mile of line. On account of the three-fold increase in the number of daily trains, the yearly mileage of the locomotives and motor cars will be much increased by the shorter delays at the terminals. Therefore, it may be safely estimated to be one-third larger than with 80 daily trains, or 97 333 instead of 73 000 miles. The cost of the moving-power equipment per mile of line, therefore, will be $\frac{3}{4}$ $\times$ 3 = $2\frac{1}{4}$ times $10 000 = $22 500.

The total cost of the plant for the direct-current operation of an average of 240 daily trains on four tracks, therefore, is as follows:

	Per mile of line.
Four 90-lb. protected third-rails	$22 400
One 0000 copper feeder	750
Track bonds	2 000
Towers	1 600
Sub-stations	16 800
Copper for high-tension lines	5 200
Total	$48 750
Moving-power equipment	22 500
	$71 250

The average efficiency from the locomotive drivers to the power-station is 0.80 $\times$ 0.963 $\times$ 0.92 $\times$ 0.97 $\times$ 0.97 = 66.6 per cent.

Alternating-Current Operation on a Four-Track Line (IV a of Tables 2, 3 and 4).—The cost of the contact conductors, their feeders and supports, is practically twice that for double track.

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With the power-stations 45 miles apart, the efficiency of the power transmission is 97.1 per cent. The total average efficiency from the drivers to the power-station is $0.971 \times 0.75 = 72.8$ per cent. Allowing for the fact that the electric locomotives and motor-car equipments are heavier than with direct-current operation, this efficiency is equivalent to $\frac{72.8}{1.07} = 68\%$ efficiency with the train weights of direct-current operation. The power furnished by the central stations, therefore, is practically the same with both methods.

Therefore, for alternating-current operation, the following is a summary of costs:

Fixed plant	$22 400
Moving-power equipment	39 375
Total	$61 775

With direct-current motors, the attendance for the sub-stations costs $533 per mile per year. Allowing for interest, taxes, maintenance and depreciation as before, the yearly cost of direct-current operation is $8 063; that of alternating-current operation is $7 334.

The foregoing estimates, *III d, III a, IV d, IV a,* give the costs per mile of regular main line, exclusive of the power-stations; they give a fair comparison of the relative costs of direct-current and alternating-current operation. However, they do not include the costs due to the railway and highway crossings, the curves, the changes required in the signaling apparatus and the equipment of side tracks. These costs vary greatly on different lines, and, therefore, were provisionally omitted from the foregoing estimates.

To obtain a roughly approximate estimate of the total cost of plant and operation, inclusive of interest and taxes, the cost per horse-power-year of the transmission plant, exclusive of rotary-converter stations, was increased 35%, and the cost of power transmission caused by the maintenance, depreciation, interest, and taxes on this plant was increased in the same ratio. The costs due to motormen on the locomotives and motor cars are not included in the foregoing detailed estimate; it is the same for both kinds of operation. If one man is used on the locomotives and motor cars, as is usual on interurban lines, this cost is estimated liberally if taken at 4½ cents per train-mile on the main tracks.

The switching operations require about 6% of the total power. The costs of and on the locomotives per horse-power-year developed in these operations are estimated to be three times as large as on the main line. This adds 12% to the costs of and on the locomotives per horse-power-year. The cost of the large steam-turbine

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power-plants here required is estimated at $120 per horse-power of average output, that of one horse-power-year produced by these plants is estimated at $40, inclusive of 5% on the cost of the plant for interest and taxes. The average power output, measured at the rim of the drivers with the 80 trains per day of case *III d* is 100.56 h.p. per mile of line, and three times as much in case *IV d*. The costs given per mile of line, therefore, can be easily changed to costs per horse-power-year at the rim of the drivers by dividing by 100.56 in case *III d* and by 301.68 in case *IV d*.

All the other cases were calculated in a similar manner. For the high-voltage direct-current operation, the power-stations were taken 25 miles apart, and the current was transmitted directly from the power-stations to the trains without sub-stations. This is only practicable where there is a large number of trains. On a single-track line, sub-stations would be required, unless a much higher voltage is used in the contact line. An over-head steel rod was used as contact conductor. This method is not at present available, as the required apparatus is not in the market.

It is instructive to subdivide the cost of installation into the three parts: Power plant, transmission plant, and moving-power equipments; and the cost of operation, inclusive of interest and taxes, into costs of power production, power transmission and power transformation from electricity to motion.

These costs per horse-power-year developed at the rim of the drivers of direct-current equipments and per 1.07 horse-power-year of alternating-current equipments are given in Tables 5 and 6.

TABLE 5.—Cost of Installation.

Type.	Power plant.	Transmission plant.	Moving-power equipment.	Total.
I d	$230	$2 982	$105	$3 317
I a	206	675	184	1 065
II d	210	660.4	120.3	990.7
II a	210	253.8	210.5	674.3
III d	198.4	542.5	111.5	852.4
III h v d	185	172.5	122.5	480.0
III a	176.4	150.4	194.9	521.7
IV d	180	198.7	83.5	462.2
IV a	176.4	100.2	146.1	422.7

Tables 5 and 6 show clearly that though the alternate-current equipments are heavy, comparatively inefficient, and expensive, their ability to use high-voltage alternating current cheapens the cost of power transmission to so large an extent that the total cost of

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production of the net hauling power required for a given traffic is less than with 700-volt direct-current equipments, in all cases, except on four-track lines with very large traffic. The reason for this is the low voltage in the contact conductor and the consequent necessity of using expensive rotary-converter sub-stations. The writer heartily agrees with Mr. Sprague on the necessity for higher voltage.

TABLE 6.—COSTS OF OPERATION INCLUSIVE OF INTEREST AND TAXES ON INSTALLATION.

Type.	Power production.	Power transmission.	Power transformation.	Total.
I d	$74.70	$298	$35.4	$407.10
I a	60	67.5	46.50	174.00
II d	64.70	64.10	41.60	170.40
II a	64.70	20.3	54.20	139.20
III d	66.10	53.70	30.20	150.00
III h v d	58.60	13.8	31.8	104.2
III a	60.00	12.0	42.0	114.00
IV d	60.00	19.3	26.30	105.60
IV a	60.00	8.0	35.10	103.10

The tables herein show the superiority of 3 000-volt direct-current operation with 80 or more trains per day on double track, and by inference still more on four-track lines. It is quite probable that still higher voltages will soon be available, and that the economic field of direct-current operation will thereby be still further increased.

Various items making up the total cost of electric operation are not yet accurately known. Among them is the cost of maintenance and depreciation of the moving-power equipment, and of the contact lines. There is also some doubt whether one or two men will be required on the locomotives. The foregoing estimates, therefore, are only rough approximations. Several important conclusions, however, may be safely drawn from this and other investigations.

First.—Electric operation is highly profitable where the traffic is very large. The saving in cost of power alone is sufficient to give, with large traffic, an ample return on the investment.

Second.—In all cases of high cost of fuel and cheap available water-powers, alternating-current operation is highly profitable even on single-track lines with any amount and kind of traffic which pays with steam-locomotive operation.

Mr. Mayer. *Third.*—With large passenger traffic, suitable electric operation is always very profitable because the increase in earnings becomes an important factor.

The experience of the next few years will give ample data to determine the uncertain elements in the cost of electric operation, and the increase in passenger earnings which it will produce. More exact estimates of the rate of return on the investment required will then be possible.

MEMOIRS OF DECEASED MEMBERS.

WILLIAM THOMAS PIERCE, M. Am. Soc. C. E.*

DIED FEBRUARY 26TH, 1906.

William Thomas Pierce was born at Leominster, Massachusetts, on February 12th, 1854, and died at Watertown, Massachusetts, on February 26th, 1906, of a complication of troubles, a valvular difficulty of the heart being the immediate cause of death. He was one of four sons of John Q. A., and Elizabeth C. Pierce, the others being named Charles Q., Henry B., and Myron E.

He received a public school education only, and his life is a good example of what a man can accomplish in America by constant and continuous effort toward a definite end. At the age of sixteen he decided upon a professional career, and chose engineering as his life work, entering the office of Ernest W. Bowditch, Landscape Gardener and Engineer, of Boston, Massachusetts, to begin at the bottom of the professional ladder.

He was always a close observer, a great student and an incessant reader, and rendered his professional studies more than usually valuable by a habit, early contracted, of card-indexing current engineering magazines, reports and textbooks. His indefatigable industry resulted primarily in his great proficiency as an engineering draftsman, and some of his pen and ink drawings turned off in the 70's might even to-day be used as the best types of that class of work.

In 1880, with the full approval and cordial co-operation of his employer, he obtained a position on the engineering force of the Mexican Central Railroad, then under construction, in which position he not only gained experience in railroad construction but familiarity with the running of preliminary and location lines. In accepting this position he was able, not only to add materially to his professional knowledge, but to broaden his general experience.

Two years later, he undertook certain preliminary and location surveys for a railroad near Quebec, Canada, expecting to be employed later on construction. In this, however, he was disappointed, the parties having charters for the roads being unable to make satisfactory financial arrangements. In 1885 he returned to Boston and re-entered the employ of E. W. Bowditch, as a First Assistant Engineer.

For the next few years most of his time was devoted to preparing schemes and superintending the construction of sewerage works; and Bar Harbor, Maine, and Waltham and Newburyport,

* Memoir prepared by Ernest W. Bowditch, Jun. Am. Soc. C. E.

Massachusetts, show the results of his care and foresight. His ingenuity and fertility in resource, about this time, were shown in the development of many original methods of pipe sewer construction and testing materials—methods which have been adopted since then by many members of the profession.

In 1893 he decided to establish a private practice where he could be more independent than was possible in a salaried position, and where he could undertake, within reasonable limits, such original investigations as he chose, without interfering with duties which had a right to claim his time. Almost immediately, however, he was appointed Superintendent of Streets, in Watertown, Massachusetts, where he lived. He retained this position until he was selected as Chief Engineer of the Metropolitan Park Systems in Boston, which post he filled until 1903, when he felt obliged to resign, owing to ill health. While in the Park employ, he was, naturally, so fully occupied that he was unable to give any time to private practice, excepting in a very limited way as a consulting engineer, so that this attempt to establish private practice proved abortive.

After retiring from the position of Chief Engineer of the Metropolitan Parks, he still continued his interest in the work of the office he had organized, and retained this until the very end, in February, 1906.

The Metropolitan office, when he took charge, was newly organized, or, rather, he organized it, systematizing the methods of keeping note and calculation books, plans, etc., so that any work, whether current or otherwise, could be easily and quickly referred to without loss of time. A detailed system of progress reports on each and every piece of construction, in a form devised by Mr. Pierce, which reports were filed periodically in the main office, not only rendered it possible to study intelligently any work either completed or in progress without being obliged to confer with the assistant or assistants having charge, but proved of inestimable value in case of dispute with contractors over payments for work, allowances for extras, and the like.

Besides this, detailed daily reports were made on construction, which were signed once each week by both contractors and inspectors. This simplified the final adjustment of any and all claims for extras and allowances, and made unnecessary the usual discussion and division on ambiguous clauses of contracts. For, when discussion is delayed until the completion of the work, after many matters of detail may have been forgotten, these frequently cause delay and sometimes lawsuits before settlement. Such reports would scarcely be practicable in the daily business routine of a private office, but, in a public office, such system and detail must prove, not only possible, but very desirable.

Though high-strung and impatient, he had himself thoroughly

under control, and, even at times of great provocation, seldom allowed himself to be caught unawares. Early in his professional life he adopted as a mental safety valve the scheme of writing out his first impressions, if pleased or annoyed by people and things. This statement he would put under his pillow at night, and almost always the next morning he was ready to modify his statements of the previous day. Later in life, he changed this practice and usually expressed no opinion whatever until he had slept on it.

Professionally, it may be said of him that he was successful to a marked degree; that he was a good workman who knew how to use his tools to best advantage, and was intolerant alike of slovenly or careless work in others.

Upright and honest in his dealings, methodical, persistent, conscientious and resourceful with his work, he was a man who had not only the entire confidence of his employees but, what is quite as important and less common, the good will of the laboring men and contractors with whom he came in contact.

In August, 1903, on account of his physical disabilities, he practically withdrew from active work, and, from that time until his death, he was scarcely ever entirely free from pain and discomfort. Even through his last sickness he remained cheerful, and ready to look on the bright side of everything, paying no attention to his own physical infirmities, and always interested in the latest engineering work, whatever it might be.

He was elected a Member of the American Society of Civil Engineers on December 5th, 1894. He was also a member of the Boston Society of Civil Engineers, the Middlesex Club, the Boston Club, and the Boston Athletic Association.

In 1883 he married Almira P. Goss, of Salem, Massachusetts, who, with one daughter, survives him.

PER BRYNN, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 10TH, 1906.

Per Brynn was born at Modum, Norway, on February 21st, 1866. He received his early education in a private school, at the place of his birth. Later, he attended the high school at Drammen, from which he was graduated with high honors in 1882. He then entered the Technological Institute at Trondhjem, from which he received the degree of M. E. in 1886. The following year was devoted to a post-graduate course in Civil Engineering in the same institution.

In 1887 Mr. Brynn came to America, and entered the service of the Shiffler Bridge Company, of Pittsburg, in the capacity of draftsman and estimator. In January, 1894, he was placed in charge of the Annex to the Company's main drawing room, where he remained until August, 1898, when he was promoted, being put in charge of the branch office in Philadelphia. In June of the following year he was recalled to Pittsburg and appointed Chief Draftsman for the Company.

When the Shiffler Bridge Company was taken over by the American Bridge Company, in May, 1900, Mr. Brynn was placed in charge of one of the structural departments of the latter company, where he remained until the time of his death.

Mr. Brynn was a man who left the impress of his personality on all who met him. One felt immediately that he was worth knowing—an impression that association only strengthened and confirmed. He was always dignified and reserved, unassuming in manners, and thoughtful of others. Sincerity of purpose and faithful effort characterized his whole work and gained for him the high regard of all who knew him.

On February 1st, 1906, Mr. Brynn was married to Mrs. Josephine Opsion, of Pittsburg. Just ten days later, in New York City, while still on their wedding trip, he passed away. He is survived by his wife, his mother and three sisters, who reside at Drammen, Norway, and two brothers, Mr. Anders Brynn, of Christiania, Norway, and Mr. Soeren Brynn, of Pittsburg.

Mr. Brynn was elected an Associate Member of the American Society of Civil Engineers on February 4th, 1903.

* Memoir prepared by James K. Lyons, M. Am. Soc. C. E.

JUSTIN BURNS, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 14TH, 1905.

Justin Burns was born in Watertown, New York, on July 28th, 1870. He was of Irish parentage. He received his early education in the Watertown public schools, and was graduated from the High School with honor when eighteen years of age. He then won a scholarship in Cornell University in a competitive examination, and received his degree of Civil Engineer in 1892.

His first work was with J. D. and T. E. Crimmins on the construction of the Third Avenue Cable Railroad, in New York City. From April, 1893, to 1897, he was Assistant Engineer with the Chicago and West Michigan Railroad, engaged in designing, inspection and construction work in the bridge department. In 1897, Mr. Burns returned to New York City, and was appointed Assistant Engineer under the Commissioner of Street Improvements of the 23d and 24th Wards, and upon the formation of the greater City of New York, in 1898, he entered the service of the newly formed Department of Bridges. He prepared designs for several viaducts and bridges in the Borough of The Bronx, and was in charge of construction of the City Island Bridge connecting City Island with the mainland at Pelham Bay Park, of the Westchester Avenue Bridge over the Bronx River, of the Brook Avenue Tunnel on the Port Morris Railroad, and of a number of smaller works.

From May, 1898, to February, 1899, Mr. Burns served with the First Regiment, United States Volunteer Engineers, in Porto Rico, as Sergeant Major, and later as Second Lieutenant. On his return from the war he resumed his work in the Department of Bridges, and in May, 1900, was appointed Assistant Engineer with the Rapid Transit Railroad Commission of the City of New York, and had charge of work from the City Hall to Canal Street, including the construction of the City Hall Loop, a heavy concrete arch under City Hall Park, and the single-track crossing under the four-track main line on Park Row.

He resigned from the Rapid Transit service in October, 1902, to go with the American China Development Company, of which William Barclay Parsons, M. Am. Soc. C. E., was Chief Engineer. His work was on the construction of the Canton-Hankow Railway, first as Division Engineer of the Sam Shui Division and later as First Assistant Engineer, and he was in direct charge of the location and construction of the railroad in South China. When this

* Memoir prepared by W. J. Boucher and J. G. Theban, Associate Members, Am. Soc. C. E.

work was suspended, in March, 1904, he returned to New York City and entered the employ of the Rapid Transit Construction Company as Resident Engineer in charge of viaduct construction.

In March, 1905, he was made Engineer-in-Charge of the reconstruction and extension of the street railway system of Memphis, Tenn., by the firm of Ford, Bacon and Davis, and, while in that city, was chosen Vice-President of the Memphis Engineering Society. In October, 1905, while on a short visit to his home in Watertown, he contracted typhoid fever, and died on November 14th, 1905.

Mr. Burns was a fine musician and an accomplished linguist. He was also interested in literary work, and, besides contributing articles on technical subjects to the engineering press, lectured before several American colleges on railroad and bridge work. His highest aim was to extend the field of his labors, and he often made personal sacrifices in order to broaden his experience. His professional work was marked by a natural ability which was enhanced by unremitting study and unflagging industry. His superiors esteemed him as an able, conscientious and reliable man; his colleagues and subordinates remember his generous help and encouragement, which often inspired them to conquer difficulties. All his friends admired the gentle kindness that accompanied his noble strength of character. His untimely death cut short a career that gave promise of honor and success. In religion, he was a Roman Catholic, and he was unmarried.

Mr. Burns was elected an Associate Member of the American Society of Civil Engineers on April 6th, 1898.

ELVER LA ZELLE SHINBUR, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 7TH, 1906.

Elver La Zelle Shinbur was born at Oakland, Nebraska, on April 3d, 1879. He received his early education at the home schools and at the Omaha High School, attended the University of Nebraska from 1897 to 1901, and received the degrees of Bachelor of Science and Civil Engineer from that institution.

During the summer of 1901 he was employed as instrumentman on grading and viaduct construction in South Omaha, Nebraska, for the Fremont, Elkhorn and Missouri River Railroad. In the fall of that year he left railroad work to go to Cuba, as Topographer under the United States Military Government, where he had charge of from four to seven reconnaissance parties engaged on a military survey of the Provinces of Puerto Principe and Santa Clara. Upon the termination of the control of the island by the United States, Mr. Shinbur was made Assistant Engineer on preliminary and location surveys and construction for the Insular Railroad Company, of Havana. After remaining with that company for a year, he went to Mexico as Resident Engineer on construction with the Vera Cruz and Pacific Railway, having charge of concrete and stone masonry, the erection of steel bridges, etc. From August, 1904, to the date of his death, Mr. Shinbur was with the United States Reclamation Service. During the autumn and winter of 1904-05 he was assigned to the Uncompahgre Valley Project, in Colorado, where he had charge of four field parties engaged in topographic work and location. In March, 1905, he was appointed Assistant Engineer in the Reclamation Service, and since April of that year was in charge of the construction of the second 50 miles of the Interstate Canal, in Wyoming and Nebraska, with headquarters at Mitchell, Nebraska.

While in Cuba, Mr. Shinbur met Miss Anna M. Leland, of Ann Arbor, Michigan, whom he married in October, 1905. They had been married scarcely a year when he contracted typhoid fever and, after a short illness of ten days, died at his home in Mitchell.

Mr. Shinbur was eminently successful in all work upon which he was engaged, and was spoken of by his superiors in terms of the highest praise. In manner always quiet and reserved, he had gained the respect and affection of the men under his direction and of all those who came in contact with him. It seemed particularly sad that this young man, after earning promotion by hard work and

* Memoir prepared by C. R. Thompson, Esq., U. S. Reclamation Service, Mitchell, Nebraska.

devotion to duty, should be stricken down when the future was so bright before him.

Mr. Shinbur was elected an Associate Member of the American Society of Civil Engineers on October 3d, 1906.

FREDERICK APPEL HAUSMAN, Jun. Am. Soc. C. E.*

DIED MARCH 6TH, 1906.

Frederick Appel Hausman was born in Allentown, Lehigh County, Pennsylvania, on August 8th, 1880. He was graduated with honorable mention, as civil engineer, from Lehigh University with the Class of 1901. While at Lehigh he was a member of the Tau Beta Pi, an honorary technical society.

During college vacations, that is, between June and September, 1899, he was Rodman for the Lehigh Engineering Company, of Allentown, Pennsylvania, and, from June to September, 1900, he served as Assistant in the Engineering Department of the Atlas Portland Cement Company, of Northampton, Pennsylvania.

After graduation in June, 1901, Mr. Hausman began work as Draftsman in the corps of the Chief Engineer of the Lehigh Valley Railroad at South Bethlehem, Pennsylvania, where he remained until January, 1902, when he accepted a position as Assistant Supervisor with the Philadelphia and Reading Railroad at Bridgeport, Pennsylvania. In this position he had full charge of much important work and handled it in a very able manner. His superiors fully appreciated his services, and very reluctantly accepted his resignation in May, 1903.

At this time, his father, Mr. W. A. Hausman, Sr., called upon him to assume the management of his brickyard at Allentown, Pennsylvania. This business demanded the entire attention of a competent and reliable man. To manage successfully a business of this kind, which operates only during the summer and autumn, requires a knowledge of how to handle men, together with business and engineering abilities. When "burning" it is often necessary for the Manager to remain at the kilns day and night, as the men generally employed in brickyards are not very reliable. In this position, Mr. Hausman acquired a thorough business training.

He remained with his father until November, 1905, when he again accepted a position as Assistant Supervisor, which was offered to him by the Philadelphia and Reading Railroad Company, at Olney, Pennsylvania. He was filling this position with credit when he contracted typhoid fever, and, after a severe illness of five weeks, he died on March 6th, 1906.

During his illness he had been asked to become City Engineer of Allentown, Pennsylvania, but, of course, could not accept it.

Mr. Hausman was a broad and a clear thinker, and a bright engineer. He had a firm sense of duty which would have shown its

* Memoir prepared by C. J. McGonigle, Esq.

effect later. His love for his family and home was carried to excess; in fact, he refused several good positions because they would have taken him too far from home.

Mr. Hausman was elected a Junior of the American Society of Civil Engineers on February 3d, 1903.

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